PB-250 427

DETENTION TANK FOR COMBINED SEWER OVERFLOW, MILWAUKEE, WISCONSIN, DEMONSTRATION PROJECT

Milwaukee Department of Public Works

Prepared for:

Municipal Environmental Research Laboratory

December 1975

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TECHNICAL REPORT DATA

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1. REPORT NO. EPA-600/2-75-071	2.
3. RECIPIENT'S ACCESSION NO.	
4. TITLE AND SUBTITLE DETENTION TANK FOR COMBINED SEWER OVERFLOW Milwaukee, Wisconsin, Demonstration Project	5. REPORT DATE December 1975 (Issuing Date)
	6. PERFORMING ORGANIZATION CODE
7. AUTHOR(S) City of Milwaukee, Wisconsin, Department of Public Works, and Consoer, Townsend & Associates*	8. PERFORMING ORGANIZATION REPORT NO.
9. PERFORMING ORGANIZATION NAME AND ADDRESS Department of Public Works Bureau of Engineers Municipal Bldg. Room 612 841 North Broadway Milwaukee, Wisconsin 53202	10. PROGRAM ELEMENT NO. 1BB034 ROAP/Task 21-ASY-077
	11. CONTRACT/GRANT NO. 11020 FAU
12. SPONSORING AGENCY NAME AND ADDRESS Municipal Environmental Research Laboratory Office of Research and Development U.S. Environmental Protection Agency Cincinnati, Ohio 45268	13. TYPE OF REPORT AND PERIOD COVERED Final
	14. SPONSORING AGENCY CODE EPA-ORD

15. SUPPLEMENTARY NOTES

*(360 E. Grand Ave, Chicago, Illinois 60611)

16. ABSTRACT

The City of Milwaukee evaluated the merits of detention tanks as a practical method for abatement of combined sewer overflow pollutional discharges from urban areas. A 3.9 million gallon combined sewer overflow detention tank was constructed to intercept overflow from a 570 acre segment of the City's combined sewer area. As part of the evaluation program, an extensive sewer and river monitoring program was conducted, utilizing eleven automated monitoring stations. The monitoring program provided data utilized with a mathematical detention tank model to evaluate performance of the project detention tank and provides a basis for other design and planning situations. Based upon approximately 5 years of data and modeling studies, detention tanks were shown to be effective in preventing a large portion of the contaminants found in combined sewer overflow from entering receiving waters. General information and methods for sizing and estimating costs of detention tanks for other areas have been developed. This information was utilized to establish preliminary cost estimates for providing similar facilities to serve the entire combined sewer area tributary to the Milwaukee River in the City.

17. KEY WORDS AND DOCUMENT ANALYSIS

a. DESCRIPTORS	b. IDENTIFIERS/OPEN ENDED TERMS	c. COSATI Field/Group
*Combined sewers *Overflows *Sedimentation Data aquisition Cost comparison Abatement--pollution Evaluation	Detention tank River model Modeling studies Influences Storm runoff	13B

PRICES SUBJECT TO CHANGE

18. DISTRIBUTION STATEMENT RELEASE TO PUBLIC	19. SECURITY CLASS *(This Report)* UNCLASSIFIED	21. NO. OF PAGES 308
	20. SECURITY CLASS *(This page)* UNCLASSIFIED	22. PRICE 9.75-2.25

EPA Form 2220-1 (9-73)

*USGPO: 1976 — 657-695/5359 Region 5-11

065035

PB 250 427

EPA-600/2-75-071
December 1975

Environmental Protection Technology Series

DETENTION TANK FOR COMBINED SEWER OVERFLOW
Milwaukee, Wisconsin, Demonstration Project

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EPA-600/2-75-071
December 1975

DETENTION TANK FOR COMBINED SEWER OVERFLOW
Milwaukee, Wisconsin, Demonstration Project

by

City of Milwaukee, Wisconsin
Department of Public Works
Milwaukee, Wisconsin 53202

and

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Project No. 11020FAU

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FOREWORD

Man and his environment must be protected from the adverse effects of pesticides, radiation, noise and other forms of pollution, and the unwise management of solid waste. Efforts to protect the environment require a focus that recognizes the interplay between the components of our physical environment--air, water, and land. The Municipal Environmental Research Laboratory contributes to this multidisciplinary focus through programs engaged in

- studies on the effects of environmental contaminants on the biosphere, and
- a search for ways to prevent contamination and to recycle valuable resources.

This Report studies the environmental effects of combined sewer overflows and evaluates a potential method for controlling the combined sewer overflows and reducing environmental contaminants.

Louis W. Lefke
Acting Director
Municipal Environmental
Research Laboratory

ABSTRACT

The City of Milwaukee evaluated the merits of detention tanks as a practical method for abatement of combined sewer overflow pollutional discharges from urban areas.

A 3.9 million gallon combined sewer overflow detention tank was constructed to intercept overflow from a 570 acre segment of the City's combined sewer area. As part of the evaluation program, an extensive sewer and river monitoring program was conducted, utilizing eleven automated monitoring stations. The monitoring program provided data utilized with a mathematical detention tank model to evaluate performance of the project detention tank and provides a basis for other design and planning situations.

Based upon approximately 5 years of data and modeling studies, detention tanks were shown to be effective in preventing a large portion of the contaminants found in combined sewer overflow from entering receiving waters. General information and methods for sizing and estimating costs of detention tanks for other areas have been developed. This information was utilized to establish preliminary cost estimates for providing similar facilities to serve the entire combined sewer area tributary to the Milwaukee River in the City.

This report was submitted in fulfillment of Project Number 11020FAP by the City of Milwaukee, Wisconsin under the partial sponsorship of the Environmental Protection Agency. Data collection was completed in November, 1972. Work was completed as of October 30, 1974.

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ACKNOWLEDGEMENTS

The cooperation and assistance received from the following persons and their organizations during the course of this study is greatly appreciated:

1. Technical Advisors

Hydroscience Inc., Water Quality Consultants: Interpretation of data; development and analysis of the mathematical river and detention tank models.

Mr. John L. Mancini
Mr. Eugene D. Driscoll

Marquette University Sanitary Engineering Laboratory under the direction of Dr. Raymond Kipp: Laboratory analyses; data collection and special studies.

2. U.S. Environmental Protection Agency

Mr. Clifford Risley, Jr., Director
Office of Research and Development
Region V
Chicago, Illinois

Mr. Darwin Wright
Chief of Control and Treatment Integration
Washington, D. C.

Mr. Albert Printz, Jr., Director
Permit Program Division
Office of Water Enforcement

Mr. Ralph Christensen, Chief
Great Lakes Demonstration Program

Mr. Ron Eng
Project Officer's Representative

3. City of Milwaukee, Wisconsin

Henry W. Maier - Mayor

Alderman William R. Drew - President of the Common Council

-Common Council

Herbert A. Goetsch - Commissioner of Public Works

Edwin J. Laszewski - City Engineer

ACKNOWLEDGMENTS
(CONT.)

Herbert D. McCullough - City Engineer (Retired)

-Department of Public Works Staff -

Mr. Ted Prawdzik, Engineer in Charge
Engineer - Sewers

Mr. Edmund Hirsch, Engineer in Charge
Sewer Engineering Division (Retired)

-Construction and Operations Personnel -

Mr. Robert Burmeister (Retired)

Mr. Franklin Gerschke

Mr. Robert Hirsch

Mr. Marvin Rutkowski

Mr. Jorgen Knudsen

-City of Milwaukee Health Department -

4. Sewerage Commission of the City of Milwaukee

   Ray D. Leary - Chief Engineer and General Manager

   Lawrence A. Ernest - Laboratory Director

5. United States Weather Service

6. United States Department of the Interior, Geological Survey, Water Resources Division

7. Consoer, Townsend and Associates

   Frederick N. Van Kirk

   Gerald I. Brask

   Robert P. Biebel

   Edwin E. Pick

SECTION I

CONCLUSIONS

PROJECT AREA

Project Area Sewer System

1. Observations made during the project study indicate that all dry weather flow entering the project area's combined sewer system is normally intercepted by the Milwaukee Sewerage Commission intercepting sewer for conveyance to the Commission Wastewater Treatment Plant.

2. Recorded sewer liquid level readings obtained at the project detention tank equipment building have indicated that the interceptor sewer has been flowing full or surcharged during storm periods, indicating that the intercepting devices in the project area and upstream are operating at or above the capacity of the sewer.

3. Since maintenance requirements for the type of combined sewer intercepting device utilized in the project area appear to be minimal and since these devices suitably control both dry weather and storm period flows as indicated in items 1 and 2 above, the devices are an effective and practical method of intercepting combined sewage.

Dry Weather Sewage Quality

1. Average values of wastewater quality parameters presented in this report, are considered to represent typical dry weather quality for the project sewer system.

2. Seasonal dry weather sewage quality variations were examined for spring, summer, and winter (those seasons when large numbers of samples were collected). Spring seasonal dry weather sewage quality show the highest contaminant concentrations of all parameters investigated except coliforms and volatile solids. Dry weather sewage contaminent concentrations during the winter are higher than summer values for all parameters except phosphates and total solids.

3. Diurnal quality fluctuations were explored using winter and summer seasonal data breakdowns. Total phosphate shows a distinct pattern in both winter and summer data, with early evening values approximately three times early morning concentrations. Total coliforms show a clear diurnal variation in winter, but not in summer. This is also the case with nitrogen and suspended solids data.

4. Chloride concentrations during the winter dry weather periods suggest an increase from approximately 100 mg/l during the early morning hours until midday to about twice this concentration during afternoon and evening hours. This effect is probably due to concentrations of chlorides in snow and ice melt entering the sewer system rather than increases in concentrations in the sanitary discharges to the sewers. Based on a dry weather flow rate of 2.0 mgd and a chloride concentration increase of 100 mg/l over background concentrations between 2:00 P.M. and 10:00 P.M., a daily flushing of approximately 600 pounds of chlorides per square mile is indicated, which is in excess of the amount of chlorides present in the sewage during non-winter months.

5. The data indicates no significant effect on dry weather sewage quality as a result of an atecedent storm. The sewage quality data indicates that infiltration is not excessive in the project area sewer system.

Dry Weather Sewage Flow

1. The flow data obtained indicates the normal average dry weather flow pattern in the test area has a range between approximately 1.7 and 2.3 mgd. At extremes, flows have been observed which range from 0.9 to 2.9 mgd. The 570 acre project area is served almost exclusively by combined sewers. The area, which is residential and commercial in character with an approximate population of 19,000, is located on the north side of the City of Milwaukee. Sewage loadings on a per capita basis are as follows:

Sewage Flow:	105 gallons per day per capita
BOD:	0.10 lb. per day per capita
Suspended Solids:	0.13 lb. per day per capita

2. Normally, maximum hourly average flows are approximately 50% greater than minimum hourly average flows during a 24 hour period. Wide variation between maximum and minimum hourly flows, common to many municipal systems, has not been observed.

3. Variations in dry weather flow, either due to diurnal or seasonal fluctuations are not considered to be a significant factor in the analysis and modeling of the storm overflows in the test area.

Wet Weather Sewage Quality

1. During storm conditions the data collected clearly demonstrate a pattern of quality variation which reflects the existence of a "first flush" condition. When all available data is considered on an averaged basis, all of the sewage quality parameters investigated demonstrate the pattern whereby concentration of the contaminant decreases with time as the storm event progresses. Certain individual storm events did not follow this pattern. However, the frequency of occurrence and magnitude of the deviation from the "first flush" pattern, are not sufficient to distort the overall picture provided by analysis of the large number of samples. In many cases when deviations from the pattern occurred in an individual storm, they could be attributed to a complex storm pattern, in which a number of radical changes in rainfall intensity occurred during the duration of what had been considered a single storm event. The major emphasis in this study was to identify the broad aspect of all storm events rather than single individual events and a clearly defined "typical" pattern has been shown.

2. During a storm event the initial concentrations of many parameters in the combined sewage are higher than average dry weather concentrations. BOD, COD, total and volatile suspended solids, chlorides, nitrates and organic nitrogen exhibit higher initial concentrations in combined sewage than in dry weather sewage flow. Average maximum concentration observed for these parameters during a storm overflow event are approximately 1.5 to 2.5 times the dry weather average values. This suggests the conclusion that the primary source of such contaminants is materials which have settled and been deposited in the sewer lines, catch basins, gutters etc. prior to the start of the storm.

3. Several combined sewage quality parameters, including ammonia, total and ortho phosphate, and fecal coliforms, all exhibit maximum concentrations during storm events which are less than dry weather averages. This suggests the quite reasonable conclusion that the primary source of such contaminants is the sewage flowing in the lines when the storm occurs which is diluted by the storm flow and that pollutant discharge of these parameters is not storm generated.

4. The data obtained during the project study period indicates that no significant correlation exists between combined sewage quality variations and specific storm characteristics, with the exception of that indicated in Item 1 above.

Wet Weather Sewage Flow (Runoff Coefficient C_r)

1. On the basis of the data analyzed, storm water runoff in the test area is characterized by a runoff coefficient C_r of 0.5. Observed ranges for C_r were 0.3 to 0.8.

2. The effect of duration of the storm event, total volume of rain per storm, rainfall intensity, and interval since antecedent storm were investigated. When data includes many storm events taken as a whole, the average coefficient, C_r, more closely approximates 0.5 indicating that no variation in C_r with any of the above parameters is justified for purposes of the study.

COMBINED SEWER OVERFLOW DETENTION TANKS

Project Detention Tank Data

1. Based on the cost data obtained during the project, estimates for various size detention tanks were developed. These construction cost estimates (based on estimated December 1973 prices) ranged from $1,500,000 for a two million gallon tank to $3,000,000 for ten million gallon tank assuming comparable design and local conditions to those at the Humboldt Avenue Facility. These estimates do not include land or right of way costs.

2. An approximate estimate of annual operating cost for a typical detention tank of the type employed for this project is $30,000. This operational cost should not be significantly dependent upon detention tank size but would be directly influenced by the number of storm events over a given period of peration.

3. The method and equipment employed at the project detention tank for handling combined sewage solid materials deposited in the tank (this basically consisted of mixer agitation equipment utilized to resuspend solids materials during tank dewatering) operated as anticipated. This method of handling the solids contained in the combined sewer overflows was established as a reliable and probably suitable alternate to separate removal of sludge from combined sewer overflow detention tanks.

4. An analysis of the data obtained during the 12 month period between November 1, 1971 and October 31, 1972 indicated that the project detention tank prevented approximately 121,000,000 gallons (out of 181,000,000 gallons) of combined sewage, approximately 100,000 pounds (out of 147,000 pounds) of BOD, and 225,000 pounds (out of 321,000 pounds) of suspended solids from being discharged to the Milwaukee River, from the 0.9 square mile project area. Removals of the other combined sewage pollutant constituents studied were also significant with the percentage removal of all parameters being in the same order of magnitude.

COMBINED SEWER OVERFLOW DETENTION TANK MODEL

1. A combined sewer overflow detention tank system model was developed. This model makes it possible to evaluate both the total quantity of storm water and pollutants resulting from storm overflows, and the quantities which can be intercepted by a detention tank. The model will have a general value in that evaluations may be made for tanks of various sizes, serving a range of drainage areas, and a variety of rainfall conditions. Analysis of model predictions versus observed quality variations in overflows leaving the detention tank has provided good verification of the validity of the model including the sedimentation portion of the model employed in the detention tank program.

2. The removal efficiency (in terms of the amount of pollutant material entering the River from the tank versus the amount of material entering the tank) of the project detention tank and detention tanks in general is directly related to tank size in relation to the size of the drainage area served. To a lesser extent, the removal efficiency of a detention tank is also affected by tank pump out rate, interceptor sewer capacity, drainage area, rainfall and runoff patterns and tank sedimentation efficiency.

Discussion of Capabilities of Detention Tanks in Control of Combined Sewer Overflow

1. Detention tanks have been shown by this project to be effective in preventing a large proportion of the contaminants found in combined sewer overflow from entering receiving waters.

2. Removal of BOD and suspended solids (in terms of percent pounds not overflowed versus pounds in tank influent) can range from approximately 30% to in excess of 80% as tank size is increased from one to six million gallons per square mile of drainage area.

3. The unit removal efficiency in terms of percent removal per unit volume decreases as tank size increases. Tank unit efficiency can range from approximately 30 to 15 percent removal per million gallons per square mile as tank size is increased from one to six million gallons per square mile of drainage area.

4. Studies evaluating detention tank removal efficiencies of BOD and suspended solids indicate that the removal due to volumetric retention is much more significant than the removals due to sedimentation. Removal due to sedimentation generally increases total removal efficiency by approximately 5% over removals due to volumetric retention alone. Thus, although some increase in overall removal effiency can be attained by designing a combined sewer overflow detention tank which permits effective sedimentation to occur, the major consideration in designing the tank to meet a selected annual average removal efficiency, that is consistent with water quality standards, is volume. Cost effectiveness considerations should concentrate on maximum volume which can be achieved at a given cost and be consistent with the needs to meet water quality standards.

Detention Tank Application

1. Abatement of discharge of combined sewer overflows for the entire City of Milwaukee is a highly complex problem. Based upon the results obtained from this project, it is evident that detention tanks can be designed to eliminate pollutional discharges from combined sewer overflows, which would cause violation of river water quality standards. Thus detention tank evaluation should be included in any feasibility study for abatement of combined sewer overflow pollutional discharges.

Although detention tanks have not necessarily been established as the only method of eliminating pollution from sewer overflows in every case, the results of this study indicated that they will be a viable and economical tool.

2. For purposes of demonstrating the cost impact of the problem, and to facilitate comparison with other methods of abatement, an approximate cost estimate has been developed for construction of thirteen detention tanks to receive flows from all combined sewer overflow points on the Milwaukee River in the City which accounts for 9 of the 27 square mile combined sewered area of the City. The approximate preliminary cost estimate for this construction is $29,500,000, based on estimated December, 1973 cost indices and including the present tank. This estimate does not include costs for land, right of way, or sewer construction which would add considerably to the cost. The preliminary estimate is not based on a detailed feasibility study for each location. It is based only on a general visual survey in the vicinity of each overflow and should be considered only as an indication of the general magnitude of cost.

It is anticipated that combined sewage pumping stations will be required at four of the thirteen locations. Based on very preliminary studies, the costs of these four pumping stations will add approximately $8,500,000 to the above cost for detention tanks.

The use of detention tanks to receive flows from all of the combined sewer overflow points along the Milwaukee River also requires the construction of sewers for interconnecting the various outfalls tributary to each of the thirteen detention tanks. Based on preliminary studies, the costs for these interconnecting sewers is estimated to be approximately $9,000,000.

While the above stated costs are apparently lower than the reported cost of some alternate plans for pollution abatement, the sewer and land costs in certain areas of the City would significantly raise the total project cost. Further, the estimate includes costs for detention tanks which were sized based on the same size per unit area as used for the Humboldt Avenue facility. Such sizing does reduce the discharge of pollutants approximately 79% on an annual basis, but does not eliminate the discharge totally. Therefore an indepth feasibility study should be made for each outlet or group of outlets to determine the best pollution abatement alternate solution for that segment of the system.

3. General construction cost information relating cost per square mile to percent removal of combined sewer overflow based on detention tanks has been developed. Based on one detention tank serving a 0.5 square mile area the capital cost per square mile varies from approximately $2,800,000 to $3,200,000 as percent removal increases from 50 to 80. Based on one tank serving a 2.0 square mile area the capital cost per square mile varies from $1,000,000 to $1,400,000 as percent removal increases from 50 to 80. These estimates do not include costs of sewers, land, right of way, contingencies, or technical services. As previously pointed out for the total Milwaukee River combined overflow solution, each particular overflow situation in Milwaukee and in other municipalities will have to be evaluated in detail to determine the most desirable pollution abatement alternate for the particular installation location. The costs developed herein can be utilized in making such evaluations.

MILWAUKEE RIVER

Milwaukee River Quality

1. In general, river water quality in the section of the Milwaukee River studied (between North Avenue and Water Street) is relatively poor and contains relatively high and variable levels of polluting material. Typically, the Milwaukee River exhibits the following characteristic concentrations based on the overall analysis of the average values observed for the periods compiled during the 5 year study period.

Temperature (°C) = 0 - 22
pH = 7.5 - 8.5
COD (mg/l) = 35 - 60
BOD (mg/l) 5 - 10
Chlorides (mb/l) = 20 - 50
Total Nitrogen (mg/l) 1 - 2
Total Phosphorus (mg/l) 1 - 2
Suspended Solids (mg/l) 20 - 50
Total Coliform (no/ml) 200 - 40,000

Significant seasonal variation in most parameter concentrations is exhibited.

2. The Milwaukee River in the area being considered, is subject to wide variations in flow which can occur over relatively short intervals.

3. "Many factors affect the water quality in Milwaukee River. Some of these factors have originated geographically outside the project study area but add to the pollutional loading of river section studied.

Because of these complexities, any effort to evaluate river quality responses to storm overflows from the combined sewer system directly using the routine river quality data obtained during the study would be highly speculative. In order to obtain a reliable quantitative assessment of the effect of combined sewer overflows on water quality in the lower reach of the Milwaukee River, reliance must be placed on the mathematical model developed during this program to characterize water quality responses.

Milwaukee River Model

1. A time variable Milwaukee River water quality model has been developed and verified by observed data. The model effectively projects quality responses in the relatively complex Milwaukee River System. The model has demonstrated its value by providing insight into river water quality influences and quality responses to these influences.

River Water Model Output and Verification

1. Significant influences on river water quality in the section of the Milwaukee River studied (North Avenue to Water Street) have been identified. These influences are briefly summarized below.

(a) Bottom Deposits - Bottom deposits in the Milwaukee River degrade water quality as measured by dissolved oxygen under all weather conditions. The degree to which benthal oxygen demand depresses dissolved oxygen varies with location and with River flow. The adverse impact increases as one proceeds downstream with oxygen depletions attributable to bottom demand at Water Street ranging from 1.0 to 2.5 mg/l. The more serious conditions occur at lower river flows.

(b) Menomonee River - The Menomonee River quality has a variable influence which at times has a substantial effect. Its adverse influence, which generally occurs during wet weather, can account for as much as 1.0 to 2.0 mg/l dissolved oxygen reduction in the vicinity of St. Paul and Water Streets. The Menomonee has a negligible effect on quality in the stretch of the Milwaukee River upstream of Cherry Street.

(c) Activity By Algae and/or Macrophytes - At times algae and/or macrophyte activity is intense and it can significantly affect dissolved oxygen levels in the River. Daily variations of 3.0 to 4.0 mg/l to as much as 7.0 mg/l of dissolved oxygen have been observed, which are attributable to photosynthetic activity. Variations in this order result in D.O. levels approaching zero at times when D.O. would otherwise be in the range of 2.0 - 4.0 mg/l

(d) Flushing Tunnel - The flushing tunnel exerts a favorable influence on river water quality most of the time. A possible exception to this, tenatively suggested by the data analysis, is that some adverse effect may be exhibited, if the tunnel is operated during high river flow periods. A scouring action can then result in release or disturbance of bottom deposits accompanied by an increase in oxygen demand. Normally however, tunnel operation can increase D.O. concentrations by approximately 3.5 mg/l at Cherry Street, 2.0 mg/l at St. Paul Avenue and 1.3 mg/l at Water Street, as well as improve the aesthetic condition of the River.

(e) Combined Sewer Overflows - Under most storm conditions, combined sewer overflows entering the River between the North Avenue Dam and the Menomonee River, exhibit a relatively minor direct effect on quality as measured by dissolved oxygen in that section of the River. However, the overflows do contribute coliform organisms, BOD and other pollutants, and these can be expected to have adverse affects on areas after the river water leaves the area of study. Solving the water quality problem in the section of the River investigated in this study will require a program which considers all of the water quality influences on the river and not simply the reduction of combined sewer overflows in the immediate area. However, present regulation and national goals regarding pollution abatement lend increased importance to abatement of combined sewer overflows.

(f) Upstream Water Quality Conditions - As previously mentioned upstream quality conditions account for an estimated 25 - 60 percent of the oxygen deficits observed in the river segments studied.

2. The studies conducted indicate that elimination of bottom deposits can have a significant impact on River dissolved oxygen levels. In terms of required time and probably cost, removal of bottom deposits is the most economical single measure that would improve the River water quality as measured by dissolved oxygen. However, due to the various sources of siltation in the River such as combined sewer overflows, urban construction and farm erosion, solids will continue to accumulate. This would make periodic dredging necessary. Elimination of a portion of these sources of siltation would, of course, reduce the frequency of solids removal operations. The problem of dredged sediment will also have to be considered and could be costly.

SECTION II

RECOMMENDATIONS

1. The effect of a single detention tank on the Milwaukee River quality is relatively small due to numerous other influences on the river. However, the operation of the Humboldt Avenue detention tank should be continued since, during a year of normal rainfall, it is estimated that the tank will prevent approximately 93,000,000 gallons (out of 124,000,000 gallons) of combined sewer overflow, approximately 79,000 pounds (out of 100,000 pounds) of BOD, and 176,000 pounds (out of 219,000 pounds) of suspended solids from entering the Milwaukee River. This indicates an operating efficiency of 79% for BOD and 80% for suspended solids.

2. It is recommended that monitoring of the operation of the detention tank be continued to provide additional data which could aid the tank operation personnel in improving the tank efficiency in reducing pollutant discharges to the River as well as being of interest to others in developing plans for similar facilities. Particular emphasis should be given to coliform, BOD and suspended solids removal efficiencies. Other important water quality parameters, such as ammonia and phosphorus, could be studied.

3. On projects which included a significant amount of automation and instrumentation systems which are critical to data collection, special maintenance provisions should be made. The following items should be given special consideration:

(a) Provisions should be made to include well-trained service personnel in the project staff, virtually on a full-time basis if data collection is to be a major function of the project.

(b) Arrangements should be made to insure adequate local stocking of spare parts.

(c) The use of backup systems for the most critical system components should be included.

4. Due to the high degree of maintenance required to keep dissolved oxygen probes operating on a continuous basis in certain river water service installations, a manual D.O. collection system should be initiated unless it is imperative that continuous data is required. As progress is made in the instrumentation development field, continuously operated probes may become more practical.

5. Consideration should be given to providing a detention tank influent measuring and sampling device at the Humboldt Avenue tank location. This will eliminate the dependence on the Humboldt and Commerce monitoring station for influent measurement and sampling.

6. Planning for improvement of the Milwaukee River quality in the area studied (between North Avenue and Water Street) should include the following major factors which influence the river water quality:

(a) Milwaukee River bottom sludge deposits.

(b) Menomonee River water quality.

(c) Algae activity and growth in the river.

(d) River flushing tunnel operation.

(e) Combined sewer overflow discharges to the river.

(f) Milwaukee River water quality upstream of the area studied.

7. Since the flushing tunnel operation has been demonstrated to be a favorable influence on river water quality, its operation should be continued. The present schedule of operation by the Sewerage Commission appears to be a desirable operational plan. Due to possible detrimental scouring action, consideration and further study should be given to cutting off the tunnel operation during high river flow periods even if D. O. levels are low. Such consideration should be made in conjunction with decisions regarding sludge deposit dredging.

8. In continuation of comprehensive planning for elimination of the causes of poor water quality in the Milwaukee River, the river model developed under this project should be expanded to include water quality influence outside of the river segment which was studied. The river model developed demonstrated its value in providing insight into conditions which influence water quality, some of which were not readily apparent. An expansion of the model could provide data upon which construction priorities could be established. That is, those pollution abatement measures to be taken could be phased in a cost effective manner such that those with the greatest river water quality improvement significance would be taken earliest in the pollution abatement program.

9. Planning to evaluate the most desirable method of abating pollution due to the discharge of combined sewer overflow should include a study of the construction of detention tanks similar to the Humboldt Avenue facility at many locations. This report has demonstrated that the cost effectiveness of detention tanks may be attractive in many instances. This should be compared to alternate combined sewer overflow pollution abatement methods. Study into treatment plant and interceptor capacity should also be included in any comprehensive plan.

10. When evaluating combined sewer overflow pollution abatement plans for a large area, detailed study involving receiving stream modeling can often be very valuable in establishing priorities. The plans, as an early part of any program, should include staged construction or other remedial measures which would have the most significant influence on improvement of receiving water quality. The plan could then include maximum exposure of connected area to combined sewer overflow storage facilities as early as possible. The area connected to these facilities on a volume per area basis should be maximized. This will normally provide the greatest total removal of pollutants.

The planning could be based on higher acreage connection per unit of volume as part of the initial storage construction program. This would result in maximizing the effectiveness of storage volumes. As the construction program proceeds, the acreage connection per unit volume could be decreased as total storage volume approaches optimum.

11. Since the dredging of bottom deposits was discontinued north of E. Buffalo Street solids deposits have been accumulating. These bottom deposits exert an oxygen demand on the River and are one of the major causes of poor river water quality in the Milwaukee River. The visual manefestations of the deposits are gasification and floating sludge solids at times. The depletion of oxygen by these deposits also creates unfavorable environment for fish and other desirable forms of aquatic life. Therefore to improve the water quality of the Milwaukee River, the bottom deposits in the River should be removed because of the benthal oxygen demand with its resultant reduction in the dissolved oxygen content in the River water. Investigations into potential dredged solids disposal problems should be an early step in planning for dredging.

SECTION III

INTRODUCTION

Combined sewers, that is, sewers which transport both sanitary wastes and storm water are prevalent throughout the county, particularly in the older urban areas. As designed and constructed, these sewers originally represented an economical solution to the problem of handling both sanitary and storm flows. All sanitary sewage and storm water was initially discharged, untreated, directly to nearby receiving waters.

As the population served by these combined sewer systems grew, the result was a corresponding increase in sanitary sewage discharged to the receiving waters. As wastewater treatment plants came into general use, an attempt was made to intercept all dry weather flows for conveyance to the plants for treatment. However, during periods of rainfall, storm water flows may be in excess of the interceptor sewer capacities. Under these conditions, an adequate outlet must be provided for the combined sewers to prevent flooding of basements and streets. To provide the necessary outlet, the combined sewers are permitted to discharge untreated dilute sewage directly to the receiving waters.

According to a 1964 U. S. Public Health Service Publication,[1] more than 1900 communities in the United States, inhabited by some 59 million people, are served by combined or by combined and separate sewerage systems. Studies[2] done in 1967 by the American Public Works Association indicated that approximately 29 percent of the nation's total sewered population is served by combined sewers.

In the 1964 Public Health Service report,[1] it is estimated that the annual overflow from these systems contains 3 to 5 percent of the total raw sewage generated within the areas served, and during storms as much as 95 percent of the untreated sewage overflows to receiving waters.

The older areas of the City of Milwaukee, like many older urban areas, are served almost exclusively by combined sewers. In 1966, the Department of Public Works of the City of Milwaukee at the direction of the Common Council applied to the Federal Water Pollution Control Administration for a combined sewer demonstration grant under the Water Quality Act of 1965. In preparing the application, City Officials envisioned the implementation of a practical, economical approach to the abatement of pollution caused by combined sewer overflow.

Early in the project planning stage it was determined that this project would be limited to the study of a detention tank to which combined sewer overflows could be diverted and detained until low flow periods when the tank contents could be pumped into intercepting sewers for conveyance to treatment facilities.

The study of the detention tank concept was undertaken with the intent of satisfying the following objectives:

1. Characterize stormwater overflows from a combined sewer system in an urban test area of Milwaukee, Wisconsin.

2. Characterize the performance of a combined sewer overflow detention tank in reducing the pollutional load to the Milwaukee River caused by rainfall runoff in the test area.

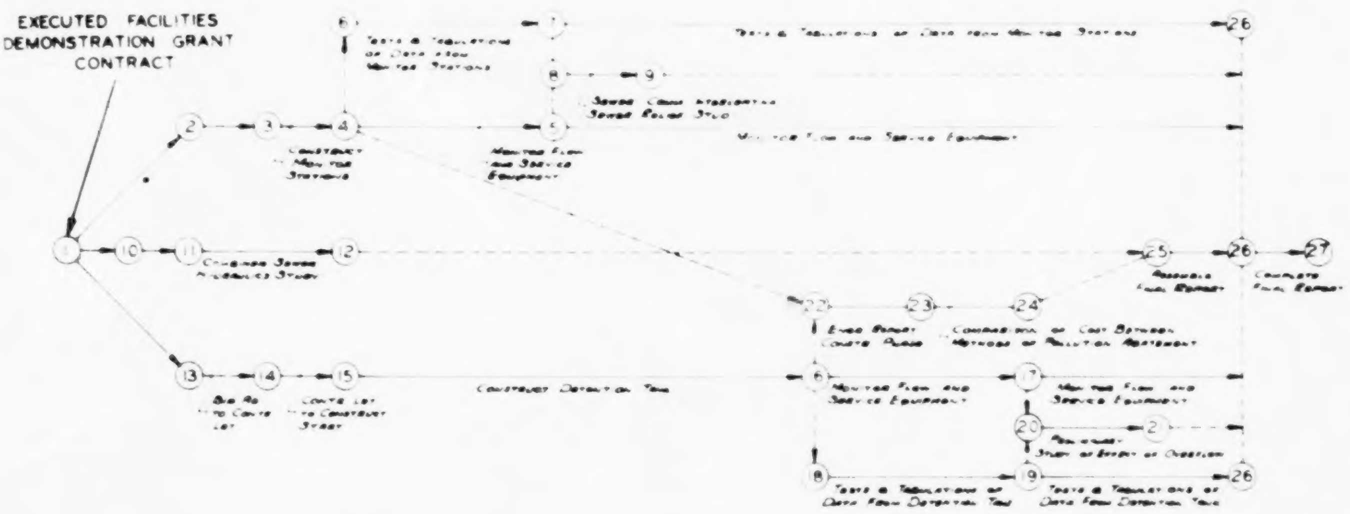
3. Evaluate cost and effectiveness of detention tanks for abatement of storm overflow pollutional discharges for urban areas larger (or smaller) than the test area studied.

4. Project the impact of combined sewer overflow detention tanks on the quality of water in the Milwaukee River.

The various phases of the project and the planned sequence of these phases is illustrated in Figure 1.

To meet these objectives a study area was defined and a thorough investigation of the study area combined sewer system and Milwaukee River, upstream and downstream from the study area, was conducted. This investigation required the design and construction of eleven sewer and river monitoring stations. Also designed and constructed was a 3.[illegible] million gallon capacity detention tank. The effectiveness of the detention tank in abating storm overflow pollutional discharges was analyzed in accordance with the project objectives.

CITY OF MILWAUKEE, WISCONSIN
HUMBOLDT AVENUE POLLUTION ABATEMENT DEMONSTRATION PROJECT


EXECUTED FACILITIES DEMONSTRATION GRANT CONTRACT
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Tests & Tabulations of Data from Monitor Stations
Tests & Tabulations of Data from Monitor Stations
Sewer Relief Study
Construct Monitor Stations
Monitor Flow and Service Equipment
Monitor Flow and Service Equipment
Combined Sewer Hydraulics Study
Contract Let
Construct Detention Tank
Comparison of Cost Between Methods of Pollution Abatement
Monitor Flow and Service Equipment
Monitor Flow and Service Equipment
Study of Effect of Overflow
Tests & Tabulations of Data from Detention Tank
Tests & Tabulations of Data from Detention Tank
Final Report
Complete Final Report

FIGURE I

PROJECT CRITICAL PATH FLOW CHART

The objectives were met during the course of the study and all findings are discussed in detail in this Report.

It is planned that the results of this Report can be utilized as part of an extensive study of the City's entire combined sewer problem. The results of such a study coupled with the existing well operated and relatively efficient wastewater treatment facilities will provide a major step in improvement of water quality in the Milwaukee metropolitan area.

SECTION IV

PROJECT DESCRIPTION

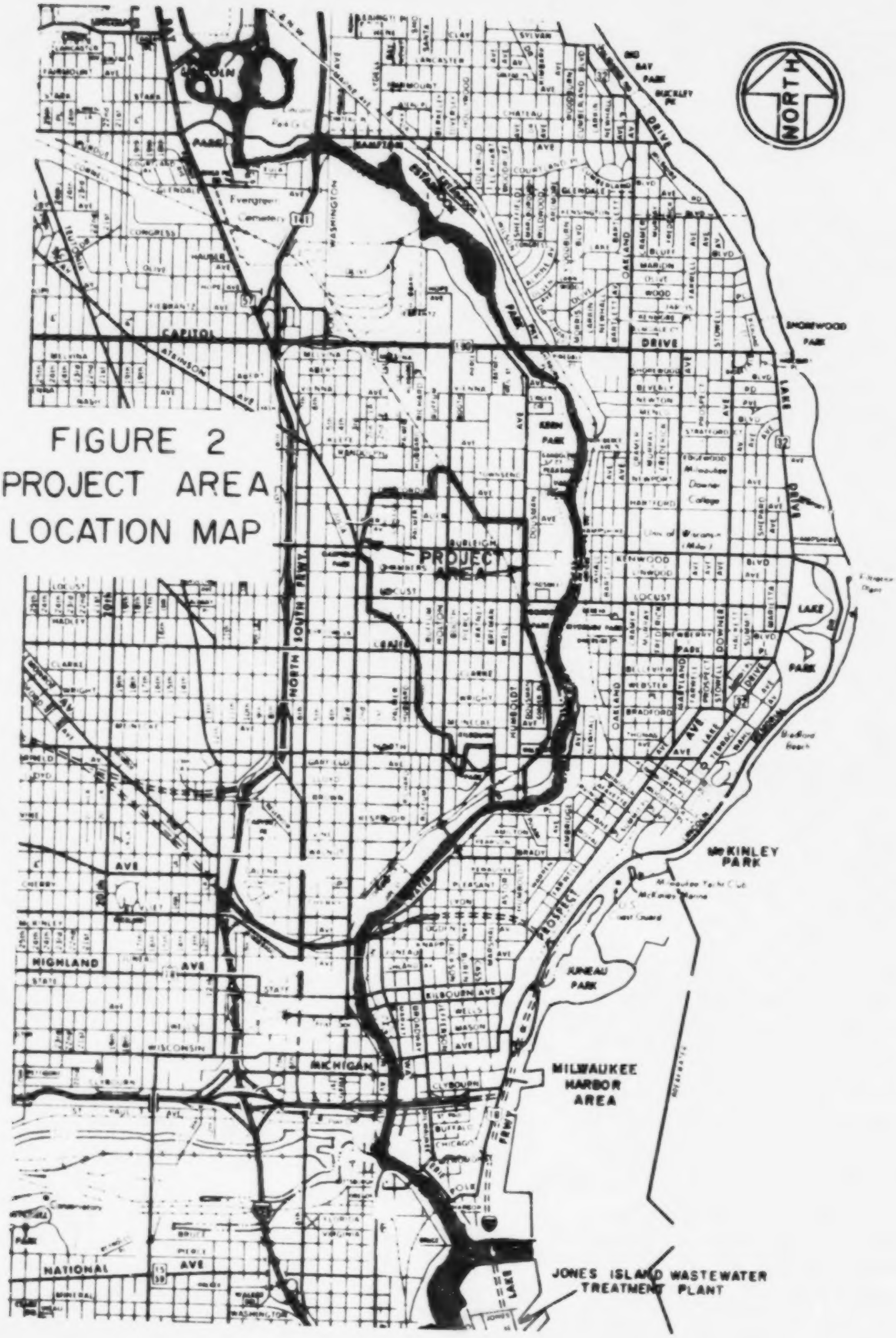
PROJECT STUDY AREA

The Humboldt Avenue Pollution Abatement Demonstration Project study area consists of 570 acres containing some 21 miles of combined service sewers and representing 1/27 of the entire combined sewered area of the City of Milwaukee. The study area is an urban section of the City's northeast side bounded generally by the Milwaukee River on the south and east sides; East Townsend Street on the north side; and North Third Street on the west. The area is residential and commercial in character and is served exclusively by combined sewers with a few separate storm sewers intercepted within the project area. The location of the project study area is shown in Figure No. 2.

PROJECT AREA COMBINED SEWERS

The combined sewer system of the project area is relieved, during periods of precipitation, at six locations by two combined relief sewers which traverse the area. These sewers also furnish relief to other local service systems adjacent to the project area. The relief sewers were constructed at elevations deep enough to permit gravity discharge from the service sewers. The overflow chambers utilized do not contain automatic regulating devices or controls. The combined relief sewers outlet to the Milwaukee River at East Locust Street and at East Auer Avenue.

Dry weather flows and a portion of the combined flow generated during periods of precipitation are directed to Milwaukee Sewerage Commission intercepting sewers for conveyance to the Commission's Jones Island Wastewater Treatment Plant. These Milwaukee Sewerage Commission intercepting sewers are referred to as M. I. S. sewers. The terms intercepting sewer and M. I. S. sewer are used interchangeably in this report. Flow to these interceptor sewers (M. I. S. sewers) is regulated by intercepting chambers (M. I. S. chambers) which divert all of the dry weather flow and a portion of the combined flow into the intercepting sewer. Flows in excess of intercepting sewer capacity enter the combined overflow downstream of the intercepting structure and discharge into the Milwaukee River.



NORTH
PROJECT AREA
CAPITOL
DRIVE
HAMPTON
WASHINGTON
Evergreen Cemetery
LOCUST
BURLEIGH
KERN PARK
NORTH SOUTH FRWY
HUMBOLDT
OAKLAND
DOWNER
Milwaukee Downer College
Univ. of Wisconsin (Milw.)
SHOREWOOD PARK
LAKE PARK
Bradford Beach
McKINLEY PARK
Milwaukee Yacht Club
McKinley Marina
U.S. Coast Guard
JUNEAU PARK
PROSPECT
HIGHLAND AVE
WISCONSIN
MICHIGAN
KILBOURN AVE
WELLS
MASON
CLYBOURN
BUFFALO
CHICAGO
MILWAUKEE HARBOR AREA
NATIONAL AVE
LAKE
JONES ISLAND WASTEWATER TREATMENT PLANT

FIGURE 2
PROJECT AREA
LOCATION MAP

The main feature of these M. I. S. chambers is the flow proportioning device which controls the amount of combined flow to the intercepting sewer. The device consists of an eccentrically pivoted flap gate mounted in a precast concrete housing set in the dividing wall between the combined sewer and the interceptor sewer. The gate is designed to rest at a maximum open position during periods of dry weather flow to permit passage of all sanitary wastes into the interceptor sewer. Experience to date is that all dry weather flow has been intercepted. As the level of flow in the combined sewer increases, due to rainfall runoff or snow melt, the increased pressure on the larger top portion of the gate causes the gate to close to a minimum open position. The gate is so designed that the increased head on this orifice and corresponding higher velocity through it result in the passage of approximately the same quantity of combined flow as the maximum permissible dry weather flow. To date recorded flow readings obtained at the project detention tank equipment building have indicated that the interceptor sewer has been flowing full or surcharged during storm periods indicating that the intercepting M. I. S. devices are operating at or above the capacity of the sewer.

Since the maintenance requirements of these M. I. S. devices is relatively minor and since they operate effectively as noted above, they appear to be an effective and economical method of intercepting flow in combined sewers and directing it to the City's wastewater treatment facilities.

There are two such intercepting structures (M. I. S. chambers) within the study area, one located on the 60 inch diameter combined sewer at North Humboldt Avenue and East Wright Street and the other on the 60 inch combined sewer at North Humboldt Avenue and North Commerce Street. The primary outlet for the study area is this 60 inch combined sewer. The overflow for this combined sewer was a 72 inch diameter overflow sewer located at North Humboldt Avenue and the Milwaukee River. It was estimated that an average of approximately 50 to 60 overflow incidents per year occurred at this point.

Further details on the various overflow and intercepting points in the project area sewer system are given in Section V of this Report.

In Table 1, the project study area combined sewer system can be compared with Milwaukee's total combined sewer system.

Table 1. MILWAUKEE-COMBINED SEWERS

	Total Municipal System	Project Area System
Combined Sewer Length in Miles	550	21
Tributary Area, Square Miles	27	0.9
Overflow and Intercepting Devices, Number:		
Overflow	109	6
Intercepting	134	2

MILWAUKEE RIVER

The Milwaukee River is a shallow, meandering stream flowing in a generally southerly direction from its source in southeastern Fond du Lac County to Lake Michigan at Milwaukee. The approximate 690 square mile Milwaukee River watershed primarily consists of agricultural, industrial, commercial, and residentially developed areas. For most of its length the river flows through soil areas carrying along large quantities of silt which results in a brownish color, characteristic of siltaceous streams.

Within the Milwaukee River watershed, the City of Milwaukee and the Village of Shorewood are the only communities served by combined sewers. The Milwaukee River Technical Study Committee noted, in their 1968 report[3] to the Mayor of the City of Milwaukee, that sources of industrial and organic pollution have, with few exceptions, been controlled north of the Milwaukee County Line. However, between Keefe Avenue and the mouth of the river there are 62 combined sewer outfalls, 52 of which discharge to the river between the North Avenue Dam and Lake Michigan. The locations and sizes of the outfalls are listed on the following Table in order of their appearance on the River beginning at Capitol Drive and proceeding south to the mouth of the River.

To monitor river flows, a discharge gaging station is maintained by the U. S. Department of Interior, Geological Survey. The station is located on the river at Estabrook Park, approximately 6 miles upstream from the river's mouth. Discharge records show that the river undergoes wide fluctuations in flow rate. The highest flow of record occurred on March 20, 1918, and August 6, 1924, when flows of 15,100 cfs were recorded and the lowest flow of record occurred on September 8, 1943, when zero flow was recorded. Minimum flows during any year may range from 0 to 200 cfs and maximum flows from 2,000 to 15,100 cfs (theoretical maximum flow has been estimated at 35,000 cfs).

RIVER FLUSHING TUNNEL

Wastewater treatment facilities were provided in Milwaukee for the first time in 1925 when the Jones Island plant was placed in operation. Prior to that time all sanitary wastes were discharged directly to Lake Michigan and the streams flowing through the City. By the late 1800's this direct discharge had transformed that section of the Milwaukee River below the North Avenue Dam into an open sewer. Unsightly surface pollution was evident throughout the year and during the summer months when temperatures are high and river flows are low the odors emanating from the river became unbearable.

Table 2. COMBINED SEWER OUTFALLS INTO MILWAUKEE RIVER

Location	Size	Location	Size
E. Capitol Drive	72"	E. Highland Avenue	36"
E. Keefe Avenue	54"	W. State Street	46"
E. Edgewood Avenue	72"	E. State Street	60"
E. Auer Avenue	84"	W. Kilbourn Avenue	36"
E. Auer Avenue	36"	W. Kilbourn Avenue	60"
E. Hampshire Street	24"	E. Kilbourn Avenue	54"
E. Burleigh Street	Dbl.	N. of W. Wells Street	30"
	9'-6" x	W. Wells Street	45"
	4'-3"	E. Wells Street	48"
E. Chambers Street	21"	N. of W. Wisconsin Ave.	18"x30"
E. Locust Street	78"	W. Wisconsin Avenue	24"
E. Park Place	60"	E. Wisconsin Avenue	30"
E. Bradford Avenue	72"	W. Michigan Avenue	54"
E. Boylston Street	72	E. Michigan Avenue	42"
E. Tunnel Place	12"	N. of W. Clybourn St.	24"x26"
N. Pulaski Street	72"	W. Clybourn Street	30"
N. Humboldt Avenue	72"	E. Clybourn Street	Dbl. 48"
N. Marshall Street	24	N. of W. St. Paul Ave.	30"
N. Holton Street	7'x4'	W. St. Paul Avenue	6'x3'
E. Grady Street	[illegible]	E. St. Paul Avenue	8'-6"x4'-0"
N. of E. Walnut Street	[illegible]	E. Buffalo Street	42"
E. Walnut Street	42	E. Chicago Street	6'x4'
E. Pleasant Street	[illegible]'x3'	S. First Street	24"
N. of W. Cherry Street	[illegible]'x4	S. Water Street	24"
W. Cherry Street	[illegible]0	E. Pittsburgh Avenue	24"
E. Lyon Street	30" &	N. Broadway	30"
	18"	S. of E. Oregon Street	30"
E. Ogden Avenue	[illegible]'x3'	E. Florida Street	60"
W. McKinley Avenue	60"	E. Polk Street	54"
W. Juneau Avenue	[illegible]2"	E. Harbor Place	30"
E. Juneau Avenue	42	E. Bruce Street	24"
N. of W. Juneau Avenue	54		
W. Highland Avenue	[illegible]'-[illegible]"		
	x4'-[illegible]		

In 1888 a tunnel was constructed between Lake Michigan and the Milwaukee River through which large volumes of lake water could be pumped to the river, discharging just below the dam. The oxygen rich lake water served to dilute the polluted river water and by increasing the dissolved oxygen content in the river the obnoxious odors were greatly reduced. It is believed that the flushing tunnel operation plus the Corps of Engineers dredging at the upper segment of the Milwaukee River estuary, which ended in the 1950's maintained the river water quality in a relatively acceptable quality from some aspects.

Until 1964, the flushing tunnel was operated by the City of Milwaukee with the following minimum schedule:

Month	Hours Per Week
May	20
June	30
July	40
August	56
September	40
October	40
November	20

This schedule was flexible and the hours of operation were readily increased whenever lower than usual water levels or extended periods of hot weather were encountered.

In 1964 the flushing tunnel operation was conveyed to the Sewerage Commission of the City of Milwaukee, which has continued the practice of river flushing to reduce odors. The Commission schedules operation of tunnel for 30 hours per week from May through October, but this schedule is readily altered in response to changing dissolved oxygen levels in the river. When the oxygen level is above approximately 5 mg/l the hours of tunnel operation are reduced but when the level falls to 3 mg/l tunnel operation is increased. If the dissolved oxygen level does not stabilize, or if it continues to drop, the tunnel is operated on a 24 hour per day basis, and up to seven days per week.

To determine the actual effect that flushing has on the water quality of the river below the North Avenue Dam, it was necessary to determine the rate of discharge of lake water through the tunnel. The method used for this determination and the results of the study are discussed in detail in Section VII of this Report.

SECTION V

PROJECT MONITORING SYSTEMS

To characterize the stormwater overflow from the project area combined sewer system, and to evaluate the effectiveness of the detention tank in reducing pollutional loads to the Milwaukee River, it was necessary to determine the quantity and quality of the combined sewage flows generated within the project study area. A full evaluation of tank effectiveness, on a per acre basis, would have necessitated the routing of all runoff in excess of intercepting sewer capacity from the project drainage area to the detention tank. This would have required the placement of bulkheads at all relief points within the system. However, the placement of bulkheads would have resulted in the flooding of basements and possibly some streets.

The alternative to the placement of bulkheads was to determine the quantity and quality of flow at each overflow point from the project area sewer system to the relief sewers. To do this, six monitoring stations were designed for construction within the study area. Two additional monitoring stations were constructed at the intercepting sewer structure sites to obtain dry and wet weather flow data. An additional three monitoring stations were constructed along the Milwaukee River to develop a profile in the river water quality.

By projecting the data obtained from these monitoring stations and the detention tank, the impact of the combined sewered area on the Milwaukee River water quality can be reproduced to provide an over-all picture or profile of the existing water quality of the river. Estimates of improvement which might be made due to combined sewer overflow facilities can also be made. One of the river monitors was located at the North Avenue Dam, upstream of the project area combined sewer outfall and detention tank; one was located downstream at the Cherry Street bridge, after a considerable additional combined sewer area is encountered; and the third station was located at the St. Paul Avenue bridge, near the outlet of the river into Lake Michigan, downstream of most of the area served by combined sewers.

The locations of the sewer and river monitor stations are shown on Figures 3 and 4. Station identification numbers and locations are as shown in Table 3.

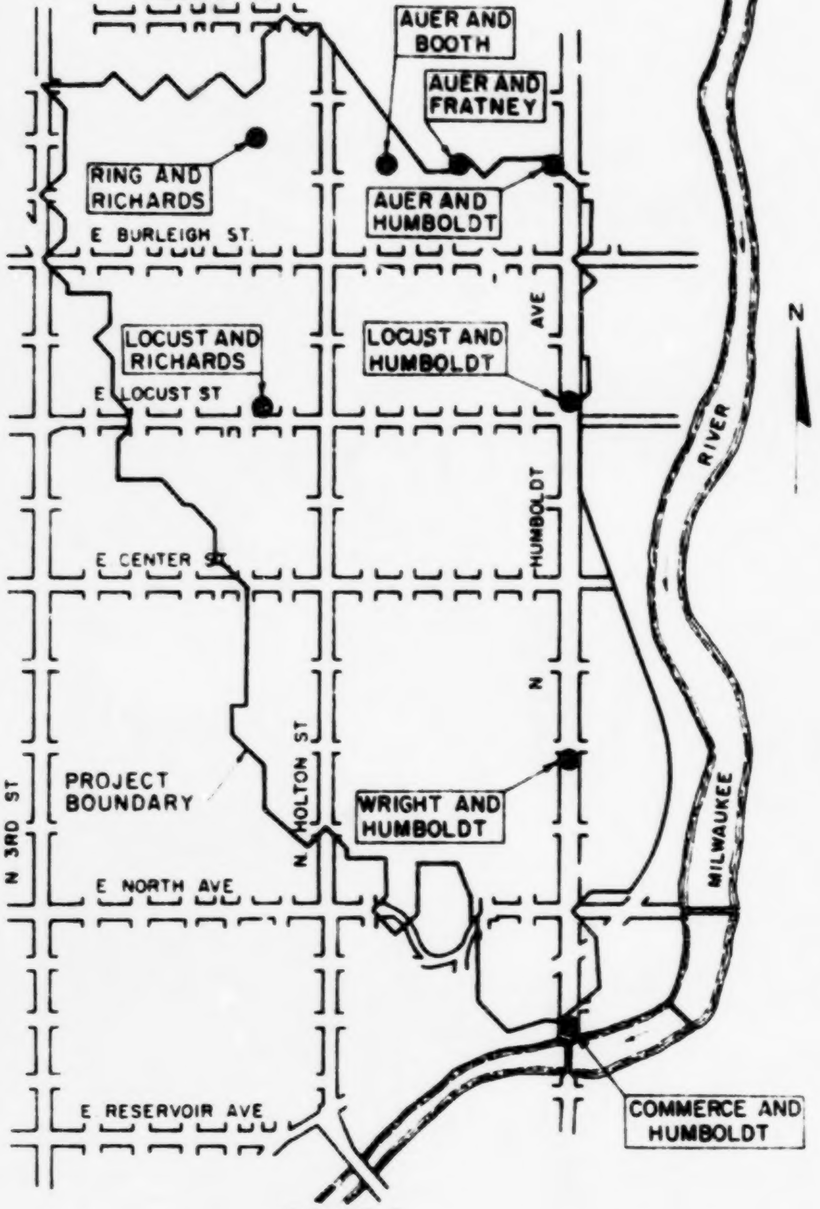
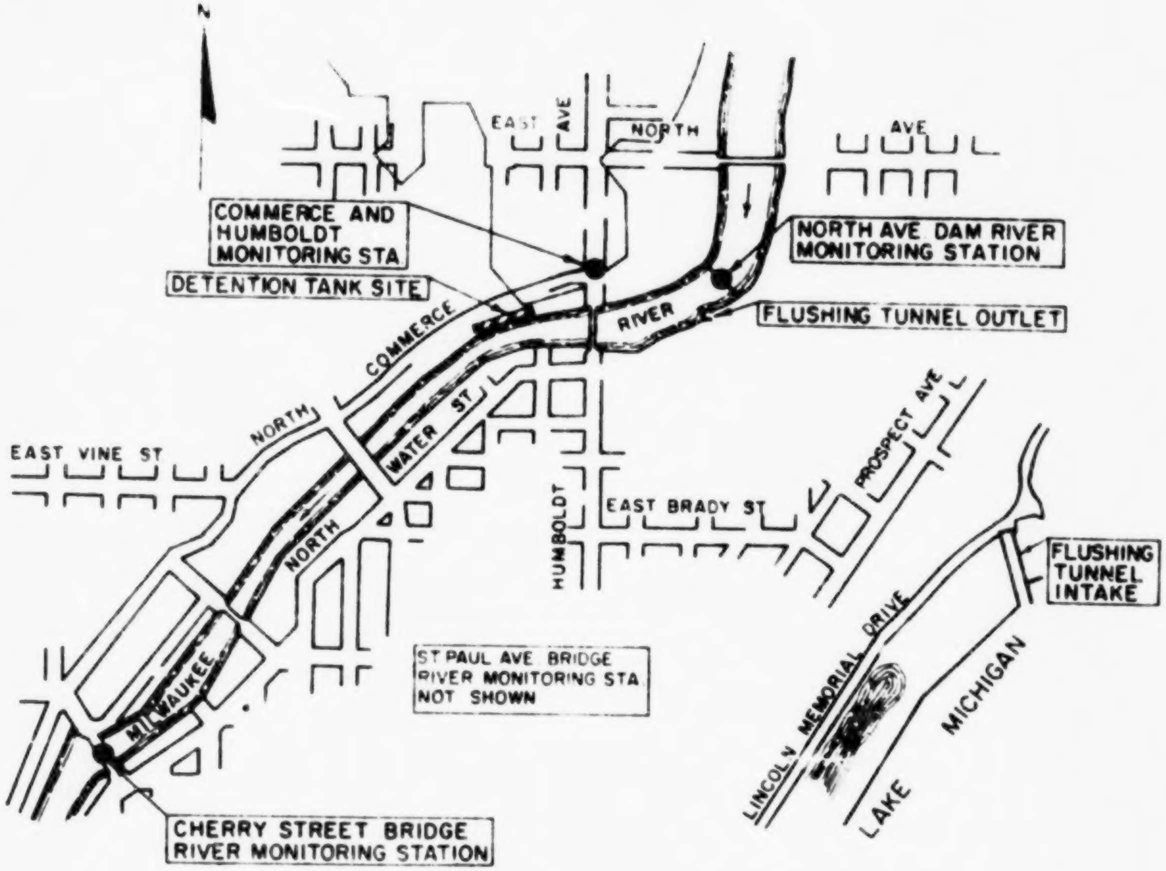

E TOWNSEND ST
AUER AND BOOTH
AUER AND FRATNEY
RING AND RICHARDS
AUER AND HUMBOLDT
E BURLEIGH ST.
LOCUST AND RICHARDS
LOCUST AND HUMBOLDT
E LOCUST ST
N HUMBOLDT AVE
N
RIVER
E CENTER ST
PROJECT BOUNDARY
N 3RD ST
N HOLTON ST
WRIGHT AND HUMBOLDT
MILWAUKEE
E NORTH AVE
E RESERVOIR AVE
COMMERCE AND HUMBOLDT

FIGURE 3

LOCATION OF COMBINED SEWER MONITORING STATIONS

NOTE: FOR FURTHER DETAIL ON THE SEWER SYSTEM AT EACH MONITORING STATION LOCATION SEE FIGURES 5,7,8,9,10 & 11



N
EAST AVE
NORTH AVE
COMMERCE AND HUMBOLDT MONITORING STA
DETENTION TANK SITE
NORTH AVE DAM RIVER MONITORING STATION
RIVER
FLUSHING TUNNEL OUTLET
COMMERCE
NORTH
EAST VINE ST
WATER ST
NORTH
HUMBOLDT
EAST BRADY ST
PROSPECT AVE
FLUSHING TUNNEL INTAKE
LINCOLN MEMORIAL DRIVE
LAKE MICHIGAN
MILWAUKEE
ST PAUL AVE BRIDGE RIVER MONITORING STA NOT SHOWN
CHERRY STREET BRIDGE RIVER MONITORING STATION

FIGURE 4-LOCATIONS OF DETENTION TANK AND RIVER MONITORING STATIONS

Table 3. MONITORING STATION IDENTIFICATION

Station Number	Station Location
Sewer Stations	
21	N. Humboldt Avenue and E. Wright Street
25	W. Ring Street and N. Richards Street
29	E. Auer Avenue and N. Fratney Street
33	E. Auer Avenue and N. Humboldt Avenue
37	N. Humboldt Avenue and E. Locust Street
41	E. Auer Avenue and N. Booth Street
45	E. Locust Street and N. Richards Street
49	N. Humboldt Avenue and N. Commerce Street
River Stations	
52	St. Paul Avenue Bridge
58	Cherry Street Bridge
64	North Avenue Dam - Downstream
66	North Avenue Dam - Upstream

The North Avenue Dam monitoring station was designed to allow monitoring of water quality both upstream and downstream of the dam.

A schematic diagram of the project's combined sewer flow metering system is included in Figure 5.

Rainfall gauging stations were also included as part of the project monitoring systems. A location map indicating the approximate location of the various pertinent project monitoring components outside of the project area is illustrated in Figure 6.

COMBINED SEWER MONITORING STATION FUNCTION

The eight combined sewer monitoring stations were designed to perform a common function; that is, to indicate, record, totalize, and sample combined sewage flow. The specific components for each station varied, but in general included the following: pressure bell type level sensing devices for activating the measuring and sampling systems; liquid level and flow measuring and recording equipment; samplers and refrigerated storage compartments; and miscellaneous piping, valves and accessories required for satisfactory operation. A description of the function and a list of the equipment for each monitoring station follows.

Station 21 - Humboldt and Wright

The function of this station was to record liquid levels in an overflow structure (M.I.S. chamber) by means of a compressed nitrogen gas bubbler; measure, indicate and record flow through a gate orifice type intercepting device (M.I.S. device) with indicating, recording and totalizing equipment; obtain samples of combined sewage in proportion to flow by an ejector type sampler with frequency control and appurtenances; and store samples in refrigerated containers.

Figure 7 shows a typical equipment arrangement used to determine quantity and quality of combined sewage flow through a gate orifice type intercepting device. A location map indicating the general location of this station and the main sewers in the area is indicated in Figure 8.

Station 25 - Ring and Richards
Station 26 - Auer and Fratney
Station 33 - Auer and Humboldt
Station 37 - Locust and Humboldt

The function of each of these four stations was to measure, indicate and record combined sewage flow through a flume or flow nozzle; record the liquid level at the crest of the primary measuring device; record the liquid level at a point upstream from the primary measuring device; obtain proportional samples of combined sewage overflow; and store samples in a refrigerated container.

FIGURE 5 – SCHEMATIC OF INPUTS TO COMBINED SEWER FLOW METERING SYSTEM

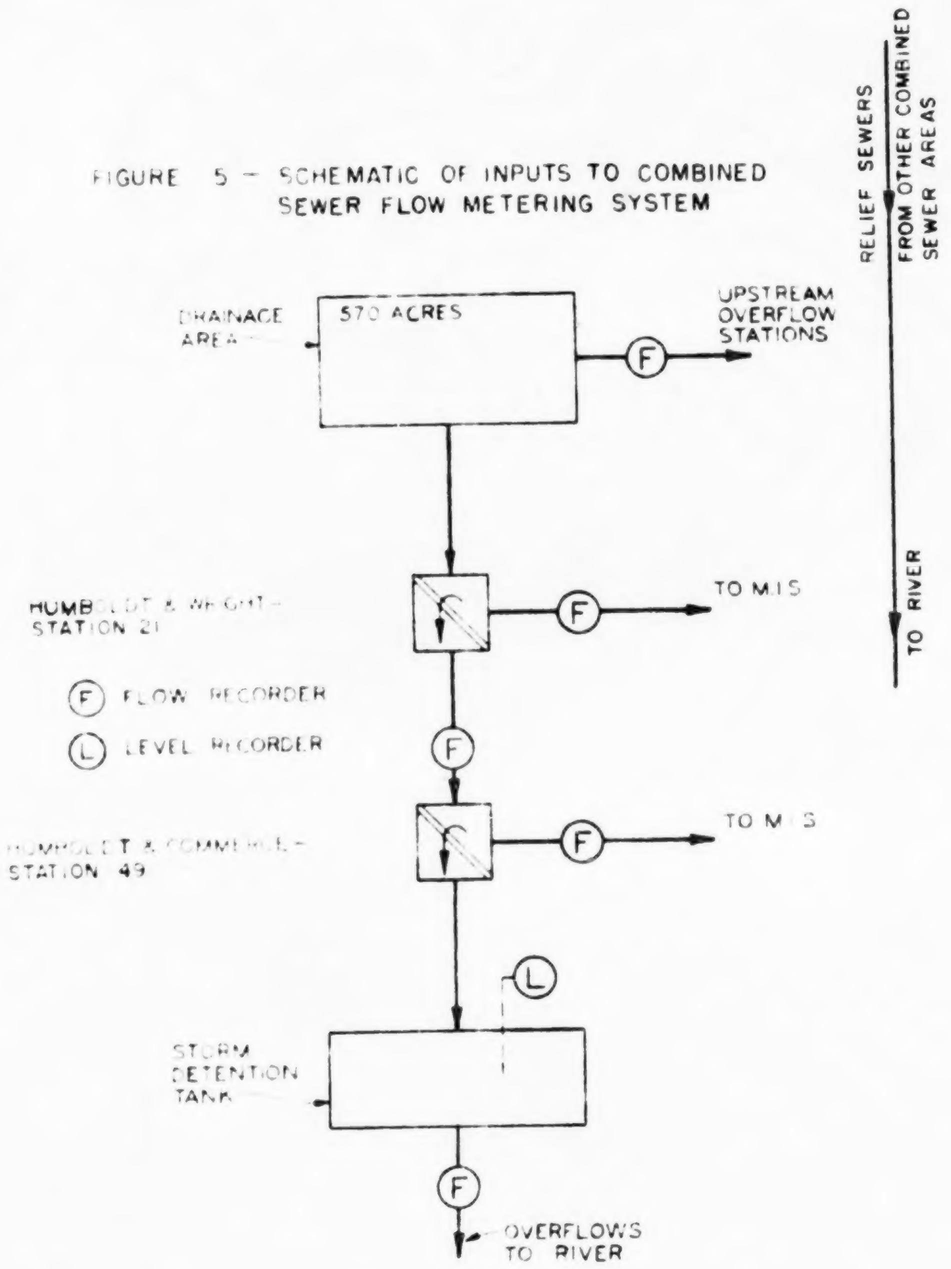
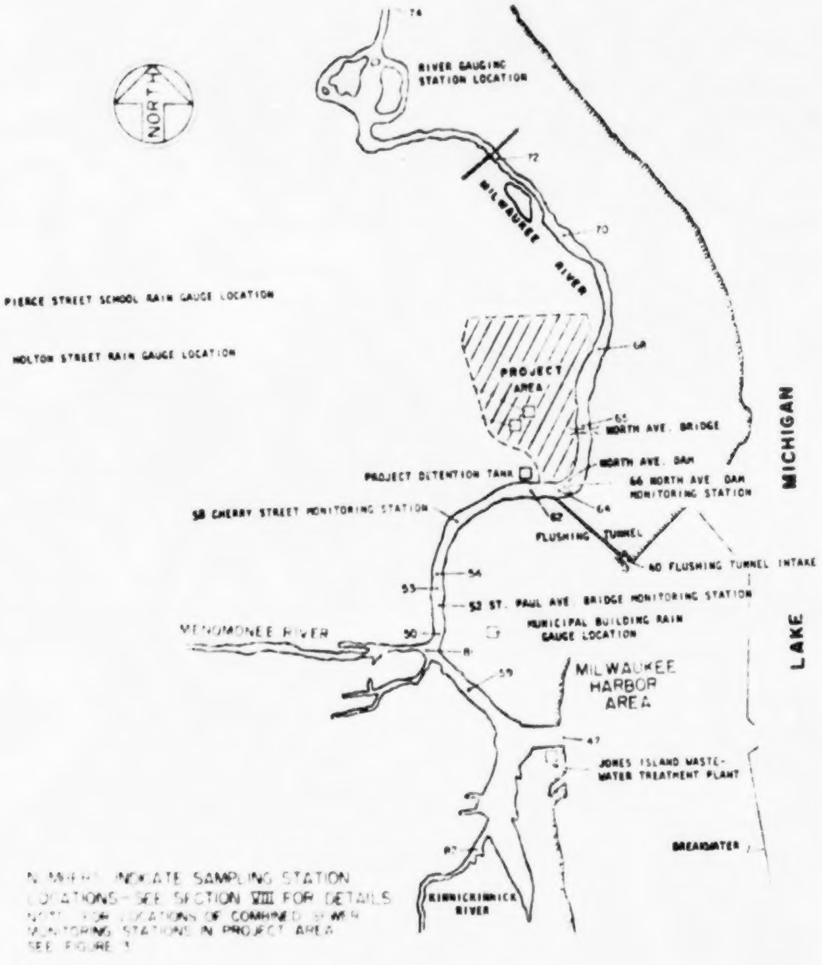

RELIEF SEWERS
FROM OTHER COMBINED SEWER AREAS
TO RIVER
DRAINAGE AREA
570 ACRES
F
UPSTREAM OVERFLOW STATIONS
HUMBOLDT & WRIGHT – STATION 21
F
TO M.I.S
F FLOW RECORDER
L LEVEL RECORDER
F
HUMBOLDT & COMMERCE – STATION 49
F
TO M.I.S
L
STORM DETENTION TANK
F
OVERFLOWS TO RIVER

FIGURE 6 - MONITORING COMPONENT
LOCATION MAP


NORTH
74
RIVER GAUGING
STATION LOCATION
72
MILWAUKEE
RIVER
70
PIERCE STREET SCHOOL RAIN GAUGE LOCATION
HOLTON STREET RAIN GAUGE LOCATION
68
PROJECT
AREA
65
NORTH AVE. BRIDGE
NORTH AVE. DAM
PROJECT DETENTION TANK
66 NORTH AVE. DAM
MONITORING STATION
64
58 CHERRY STREET MONITORING STATION
62
FLUSHING TUNNEL
60 FLUSHING TUNNEL INTAKE
54
53
52 ST. PAUL AVE. BRIDGE MONITORING STATION
MENOMONEE RIVER
50
MUNICIPAL BUILDING RAIN
GAUGE LOCATION
81
59
MILWAUKEE
HARBOR
AREA
47
JONES ISLAND WASTE-
WATER TREATMENT PLANT
BREAKWATER
87
KINNICKINNICK
RIVER
MICHIGAN
LAKE

NUMBERS INDICATE SAMPLING STATION
LOCATIONS - SEE SECTION VIII FOR DETAILS
NOTE: FOR LOCATIONS OF COMBINED SEWER
MONITORING STATIONS IN PROJECT AREA
SEE FIGURE 3

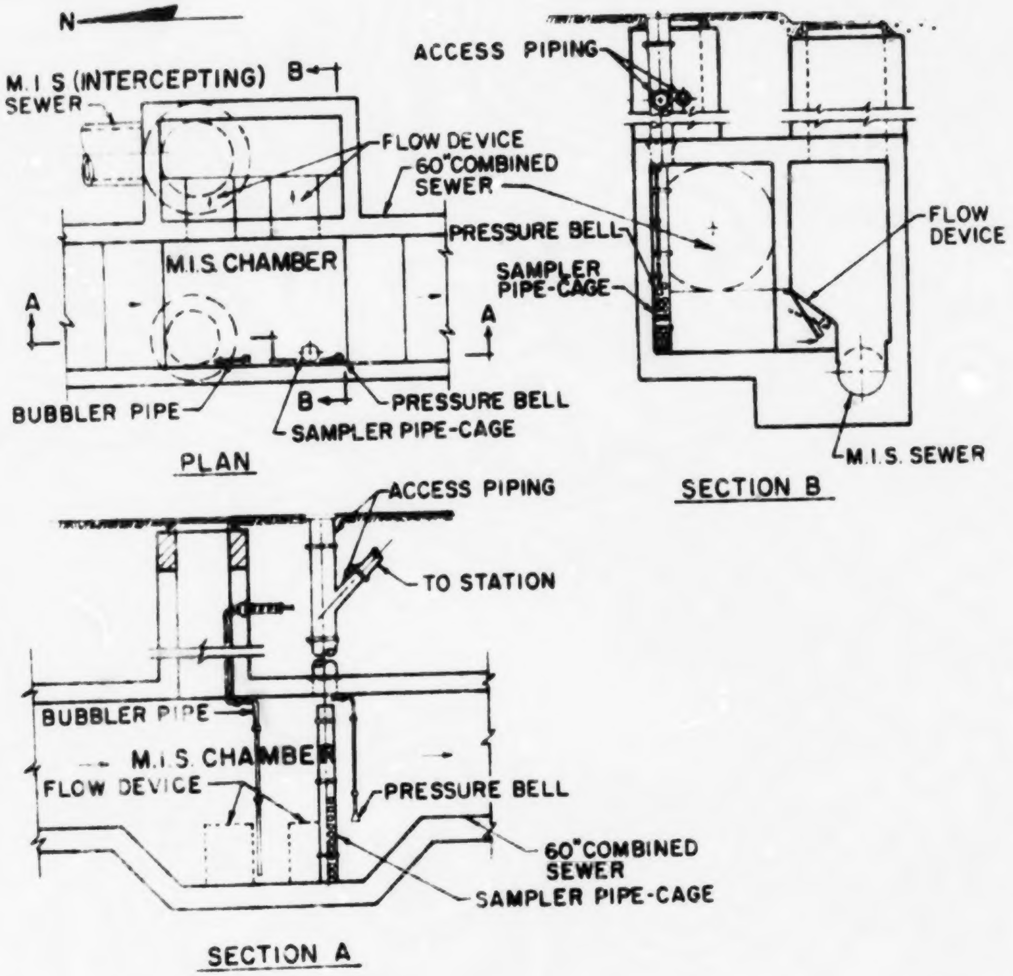

N
M.I.S (INTERCEPTING) SEWER
B
FLOW DEVICE
60" COMBINED SEWER
M.I.S. CHAMBER
A
A
B
BUBBLER PIPE
PRESSURE BELL
SAMPLER PIPE-CAGE
PLAN
ACCESS PIPING
PRESSURE BELL
SAMPLER PIPE-CAGE
FLOW DEVICE
M.I.S. SEWER
SECTION B
ACCESS PIPING
TO STATION
BUBBLER PIPE
M.I.S. CHAMBER
FLOW DEVICE
PRESSURE BELL
60" COMBINED SEWER
SAMPLER PIPE-CAGE
SECTION A

FIGURE 7

MONITORING STATION WITH GATE ORIFICE FLOW DEVICE-M I.S. CHAMBER

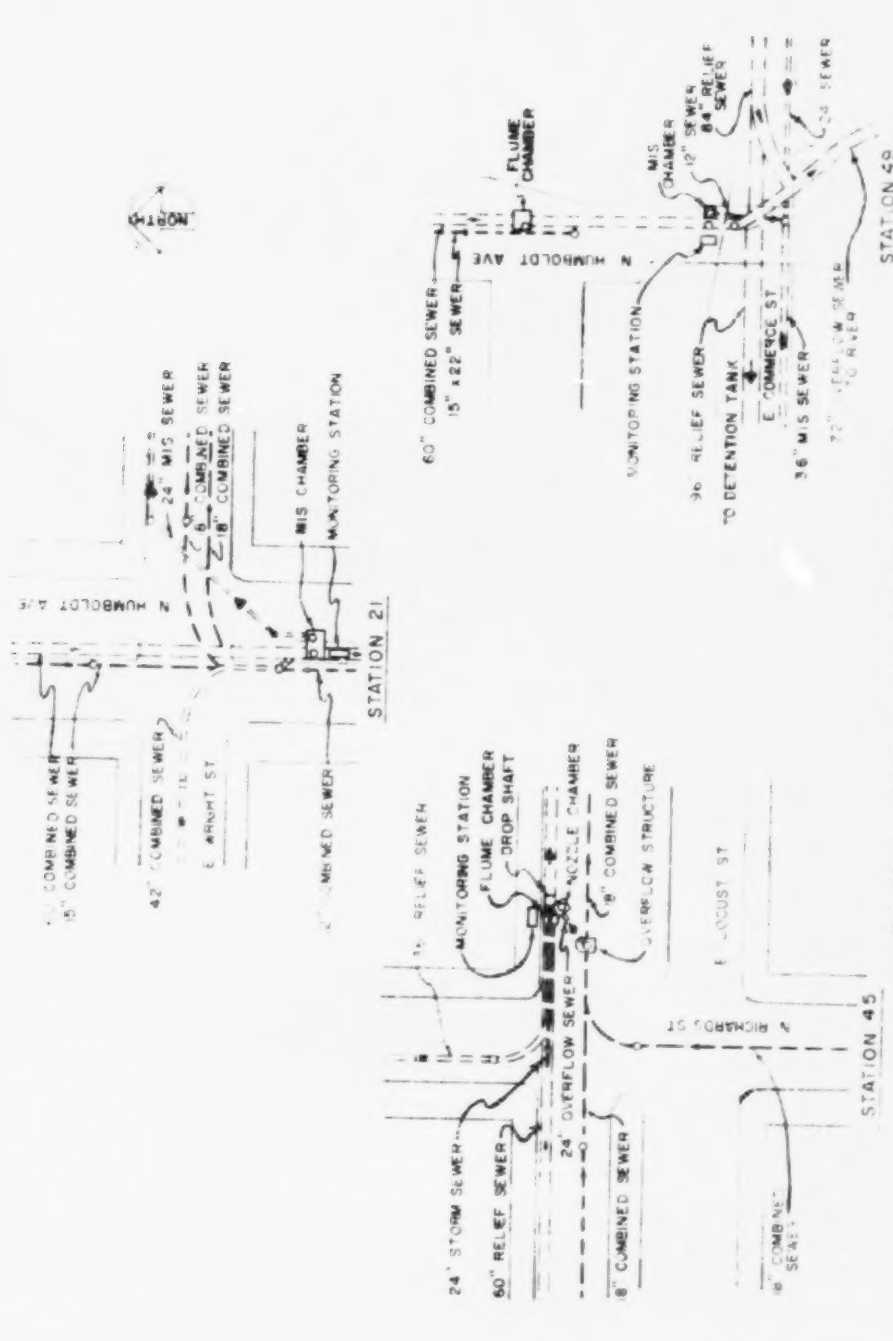

NORTH
STATION 21
COMBINED SEWER
15" COMBINED SEWER
N HUMBOLDT AVE
42" COMBINED SEWER
E WRIGHT ST
COMBINED SEWER
24" MIS SEWER
COMBINED SEWER
18" COMBINED SEWER
MIS CHAMBER
MONITORING STATION
STATION 45
24" STORM SEWER
60" RELIEF SEWER
24" OVERFLOW SEWER
18" COMBINED SEWER
RELIEF SEWER
MONITORING STATION
FLUME CHAMBER
DROP SHAFT
NOZZLE CHAMBER
18" COMBINED SEWER
OVERFLOW STRUCTURE
E LOCUST ST
N RICHARDS ST
18" COMBINED SEWER
STATION 49
60" COMBINED SEWER
15" x 22" SEWER
FLUME CHAMBER
N HUMBOLDT AVE
MONITORING STATION
96" RELIEF SEWER
TO DETENTION TANK
E COMMERCE ST
36" MIS SEWER
72" OVERFLOW SEWER TO RIVER
MIS CHAMBER
12" SEWER
84" RELIEF SEWER
24" SEWER

FIGURE 8- LOCATION MAP-STATIONS 21,45, & 49

All equipment in these stations, other than the primary flow measuring device, was similar to that used in the Wright and Humboldt Station.

Figures 9 and 10 show a typical equipment arrangement used to determine quantity and quality of combined sewage flow through a flume or nozzle. A location map indicating the general location of these stations and the main sewers in the area are shown in Figures 9, 10, and 11.

Station 45 - Locust and Richards

The function of this station was to measure, indicate, totalize and record flows through a 36 inch flume, as well as through a 24 inch flow nozzle; record the liquid level at a point upstream from each of the primary measuring devices; obtain combined sewage samples in proportion to the flow through one of the primary devices; and store samples in a refrigerated container. Two points of measurement were required because two combined sewers are relieved at this location. It was assumed that the overflow characteristics would be similar, thus sampling facilities were provided for only one of the sewers.

This station included equipment as described for the stations in preceding paragraphs, except that two sets of flow indicating, recording and totalizing equipment were required as well as four liquid level recording devices.

Station 41 - Auer and Booth

The function of this station was to measure, indicate, record and totalize flow through each of two flumes; record the liquid level at the crest of each of the flumes; record the liquid level at a point upstream from one of the flumes; obtain combined sewage samples in proportion to flow through one of the flumes; and store samples in a refrigerated container.

The station included equipment as described in the paragraph above for the monitoring station at Locust and Richards, except that only three liquid level recording devices were required.

A location map indicating the general location of this station and the main sewers in the area is shown in Figure 11.

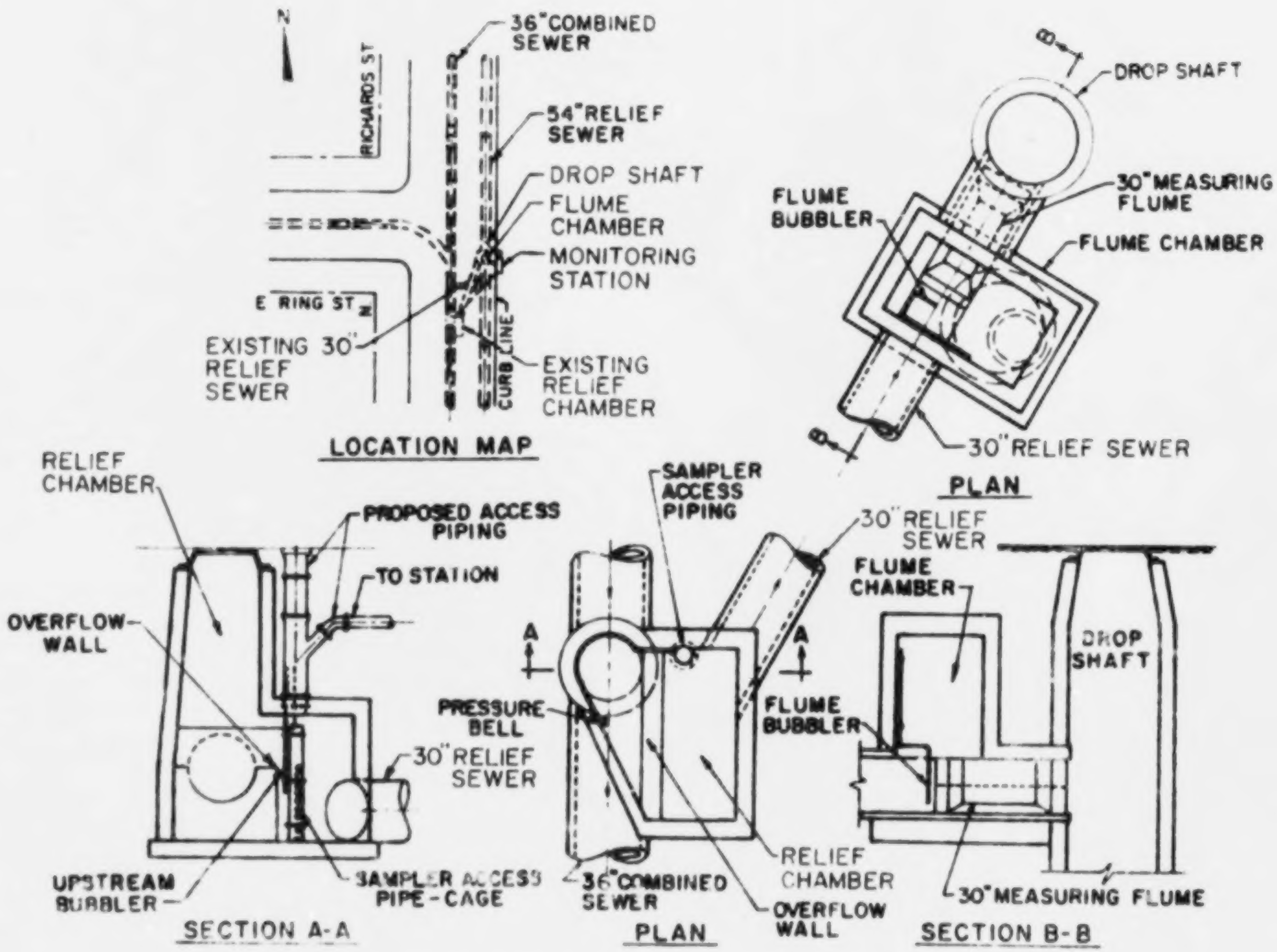
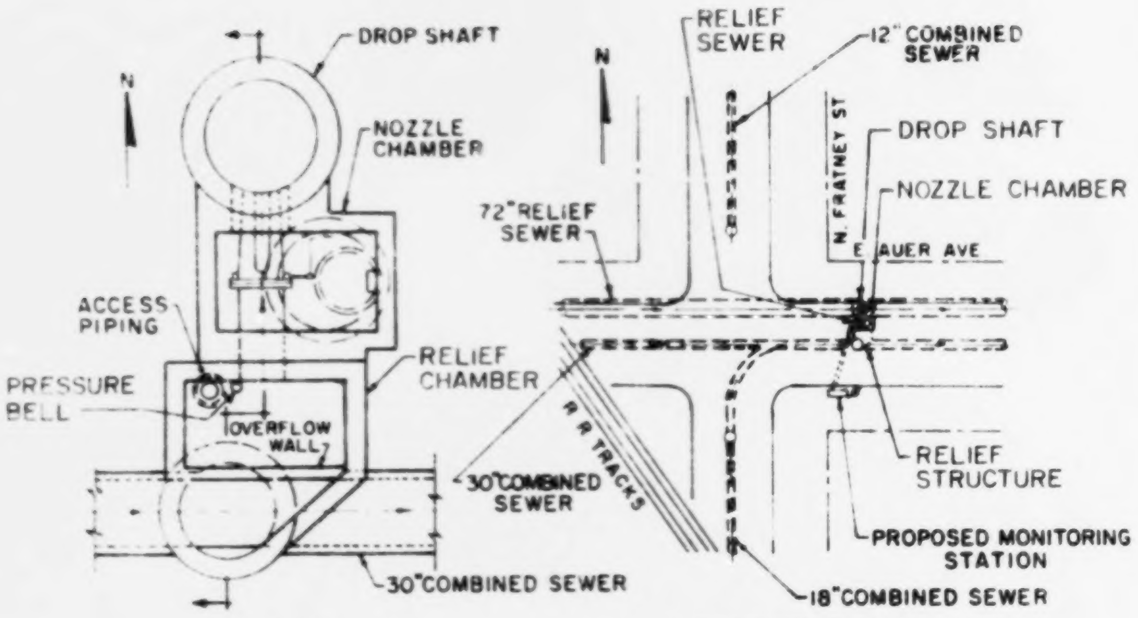
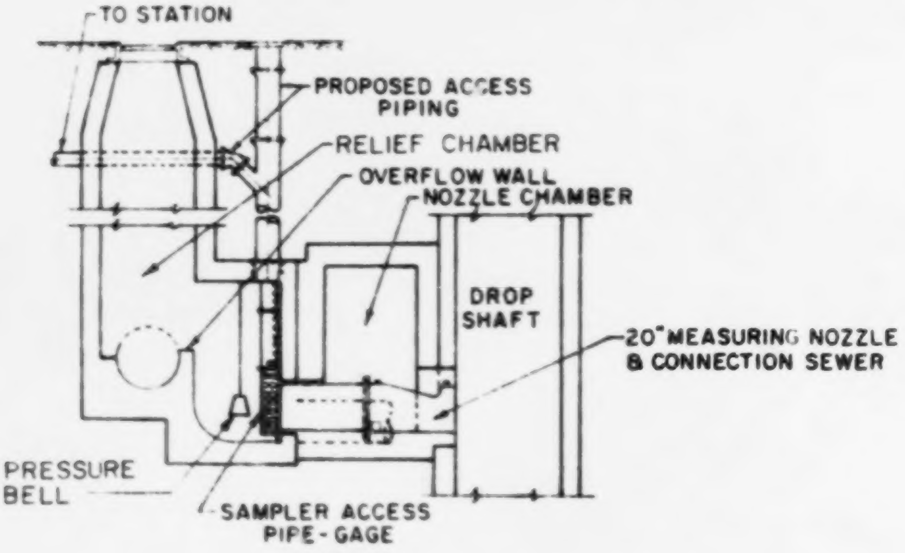

N
RICHARDS ST
E RING ST
36" COMBINED SEWER
54" RELIEF SEWER
DROP SHAFT
FLUME CHAMBER
MONITORING STATION
EXISTING 30" RELIEF SEWER
CURB LINE
EXISTING RELIEF CHAMBER
LOCATION MAP
B
DROP SHAFT
FLUME BUBBLER
30" MEASURING FLUME
FLUME CHAMBER
30" RELIEF SEWER
PLAN
RELIEF CHAMBER
PROPOSED ACCESS PIPING
TO STATION
OVERFLOW-WALL
30" RELIEF SEWER
UPSTREAM BUBBLER
SAMPLER ACCESS PIPE-CAGE
SECTION A-A
SAMPLER ACCESS PIPING
A
PRESSURE BELL
36" COMBINED SEWER
RELIEF CHAMBER
OVERFLOW WALL
PLAN
30" RELIEF SEWER
FLUME CHAMBER
FLUME BUBBLER
DROP SHAFT
30" MEASURING FLUME
SECTION B-B

FIGURE 9 - MONITORING STATION WITH FLUME FLOW DEVICE
STATION 25


DROP SHAFT
N
NOZZLE CHAMBER
ACCESS PIPING
PRESSURE BELL
OVERFLOW WALL
RELIEF CHAMBER
30" COMBINED SEWER
RELIEF SEWER
12" COMBINED SEWER
N
N. FRATNEY ST
DROP SHAFT
NOZZLE CHAMBER
72" RELIEF SEWER
E. AUER AVE
R R TRACKS
30" COMBINED SEWER
RELIEF STRUCTURE
PROPOSED MONITORING STATION
18" COMBINED SEWER

PLAN

LOCATION PLAN


TO STATION
PROPOSED ACCESS PIPING
RELIEF CHAMBER
OVERFLOW WALL
NOZZLE CHAMBER
DROP SHAFT
20" MEASURING NOZZLE & CONNECTION SEWER
PRESSURE BELL
SAMPLER ACCESS PIPE-GAGE

SECTION

FIGURE 10 - MONITORING STATION WITH NOZZLE FLOW DEVICE
STATION 29

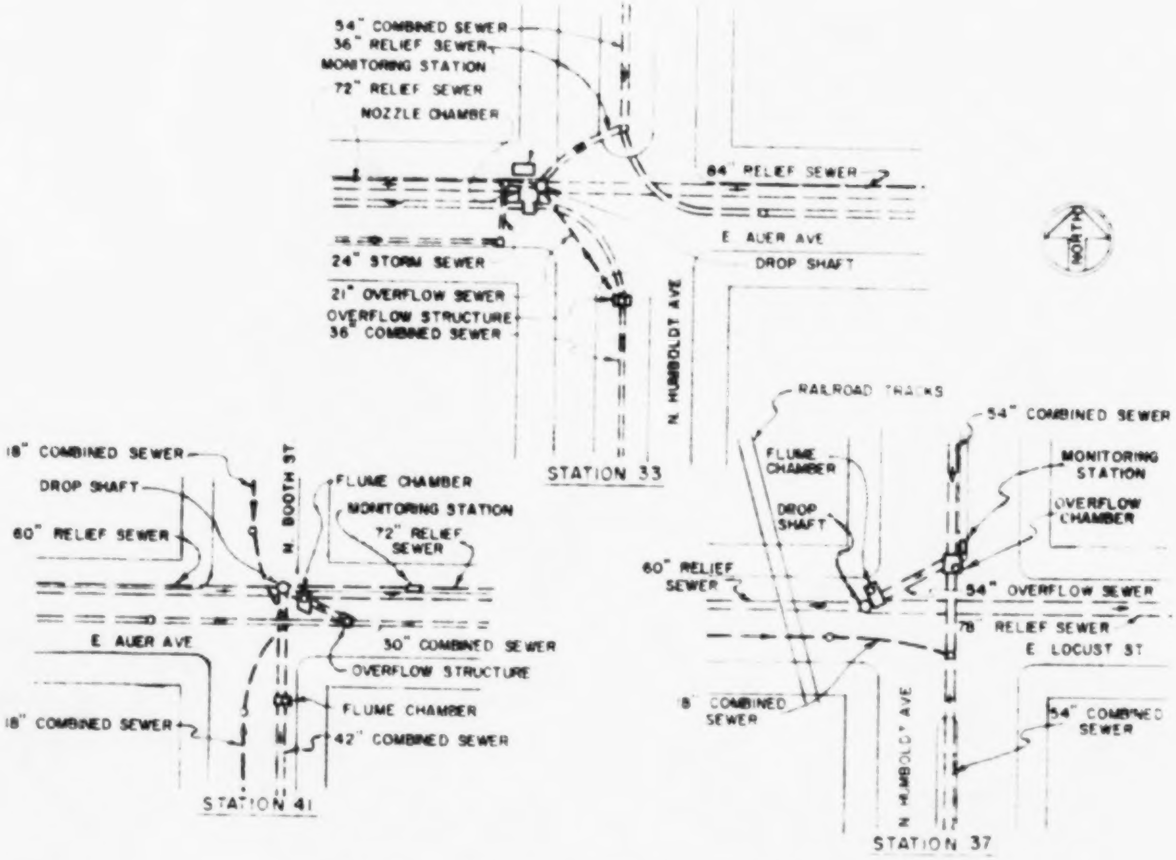

54" COMBINED SEWER
36" RELIEF SEWER
MONITORING STATION
72" RELIEF SEWER
NOZZLE CHAMBER
84" RELIEF SEWER
E AUER AVE
DROP SHAFT
24" STORM SEWER
21" OVERFLOW SEWER
OVERFLOW STRUCTURE
36" COMBINED SEWER
N. HUMBOLDT AVE
STATION 33
NORTH
RAILROAD TRACKS
54" COMBINED SEWER
FLUME CHAMBER
MONITORING STATION
DROP SHAFT
OVERFLOW CHAMBER
60" RELIEF SEWER
54" OVERFLOW SEWER
78" RELIEF SEWER
E LOCUST ST
8" COMBINED SEWER
N HUMBOLDT AVE
54" COMBINED SEWER
STATION 37
18" COMBINED SEWER
DROP SHAFT
N. BOOTH ST
FLUME CHAMBER
MONITORING STATION
60" RELIEF SEWER
72" RELIEF SEWER
E AUER AVE
30" COMBINED SEWER
OVERFLOW STRUCTURE
18" COMBINED SEWER
FLUME CHAMBER
42" COMBINED SEWER
STATION 41

FIGURE 11- LOCATION MAP - STATIONS 33, 37 & 41

Station 49 - Humboldt and Commerce

The function of this station was to measure, indicate, record and totalize flow through a flume; record the liquid level at the crest of the flume; record the liquid level at a point upstream from the flume; record the liquid level at an existing gate orifice; indicate, record and totalize the flow through the gate orifice, obtain combined sewage samples in proportion to flow through the flume; store samples in a refrigerated container; obtain additional individual samples on a time interval basis using a separate sampler, and store such samples in refrigerated bottles.

This station was located immediately upstream from the detention tank and measured and collected samples of all combined sewage flows discharging to the tank. To determine the quality of the combined sewage, at various intervals of storm flows, an additional sampler was provided at this station which collected individual grab samples at predetermined time intervals.

All equipment in this station was similar to that used in the Humboldt and Wright Station, except for the addition of a vacuum type sampler for obtaining the grab samples and a flume measuring device.

A location map indicating the general location of this station and the main sewers in the area is shown in Figure 5.

COMBINED SEWER MONITORING STATION COMPONENT EQUIPMENT

Primary Measuring Devices - General

The type of primary measuring device used at each station depended upon the length and slope of the connecting sewer between the point of overflow and the relief sewer. Where a sufficient straight run of sewer existed, a flume was used; where an insufficient straight run of sewer existed, an open flow nozzle was used. At two monitoring locations, Humboldt and Wright and Humboldt and Commerce, dry weather flow and a portion of the combined sewage flow is diverted through orifice type M.I.S. intercepting devices to the Sewerage Commission interceptor sewer. The flow of sewage to the interceptor is regulated by these devices and is maintained at a fairly constant rate by an eccentrically pivoted orifice gate, the opening of which is controlled by the depth of flow in the intercepting structure. A head measurement upstream from the orifice was used to determine and record the approximate flow through the orifice to the interceptor by means of a cam-type recorder instrument.

Under open flow conditions, it was only necessary to measure the head on either the flume or the nozzle. However, at high flows, when the connecting sewer was surcharged, it was also necessary to measure the upstream head. This information, compared with the downstream reading, provided approximate head loss data for use in calculating the quantity of flow in the sewer by use of hydraulic formulae. During these high flow conditions, flows are calculated manually, rather than automatically provided through the instrumentation system.

Figure 12 is a schematic diagram showing the interconnection of the various components of a combined sewer monitoring station.

Flumes

The flumes necessary for the required flow measurements were designed in accordance with the requirements shown in Table 4.

Table 4. FLUME DESIGN DATA

Location	Sewer Diameter	Capacity
Ring and Richards	30"	13 cfs
Auer and Booth	36"	21 cfs
Auer and Booth	42"	31 cfs
Locust and Richards	36"	21 cfs
Humboldt and Locust	54"	60 cfs
Humboldt and Commerce	60"	77 cfs

The capacities shown on the foregoing tabulation were nominal and were established by the flume manufacturer for heads equal to that resulting from a depth of flow in the upstream sewer of about 85 percent of sewer diameter. The flumes were further calibrated for heads resulting from a full depth of flow in the upstream sewer. Flows exceeded flume capacity on a few occasions.

The flumes were designed for permanent installation in a half section of sewer, and were installed with appropriate anchorage devices. The entrance and discharge ends of the flumes have a semi-circular invert section of the same diameter as the sewer in which each is installed. The flumes were manufactured of a corrosion-resistant resin reinforced with fiberglass mat to provide a minimum wall thickness of 1/8 inch throughout. Each flume was designed to produce metering heads within 2 percent of the theoretical rating curve for the physical conditions existing at each location.

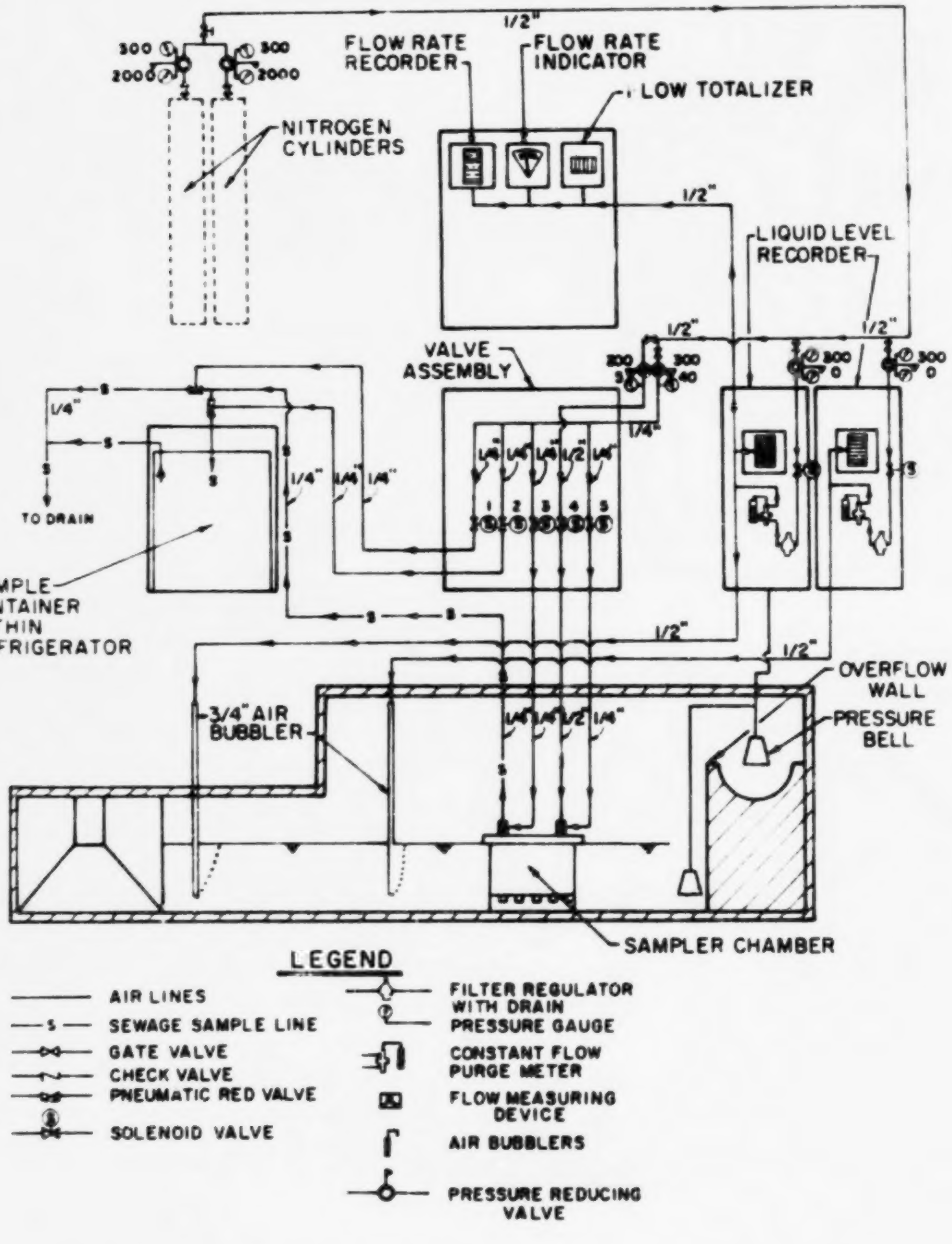

1/2"
300
2000
300
2000
FLOW RATE RECORDER
FLOW RATE INDICATOR
FLOW TOTALIZER
NITROGEN CYLINDERS
1/2"
LIQUID LEVEL RECORDER
1/2"
1/2"
VALVE ASSEMBLY
200
300
40
300
0
300
0
1/4"
1/4"
1/4"
1/4"
1/4"
1/2"
1/4"
1/4"
1/4"
1/4"
1 2 3 4 5
TO DRAIN
SAMPLE CONTAINER WITHIN REFRIGERATOR
1/2"
1/2"
OVERFLOW WALL
PRESSURE BELL
3/4" AIR BUBBLER
1/4"
1/4"
1/2"
1/4"
SAMPLER CHAMBER
LEGEND
AIR LINES
SEWAGE SAMPLE LINE
GATE VALVE
CHECK VALVE
PNEUMATIC RED VALVE
SOLENOID VALVE
FILTER REGULATOR WITH DRAIN
PRESSURE GAUGE
CONSTANT FLOW PURGE METER
FLOW MEASURING DEVICE
AIR BUBBLERS
PRESSURE REDUCING VALVE

FIGURE 12 - SCHEMATIC FLOW DIAGRAM FOR MONITORING STATIONS

Open Flow Nozzle

The open flow nozzles necessary for required flow measurements were designed in accordance with the following data:

Table 5. NOZZLE DESIGN DATA

Location	Pipe and Nozzle Diameter	Capacity
Auer and Fratney	20"	7.0 cfs
Auer and Humboldt	36"	31.0 cfs
Locust and Richards	24"	11.6 cfs

The nozzles were calibrated for flows up to the nozzle head which resulted from full depth of gravity flow in the upstream sewer section. The contour of the cast iron nozzle causes essentially equal changes in nozzle head loss for corresponding changes in flow. The change in head is sensed at a bronze piezometer opening to which is attached a gas bubbler pipe.

The manufacturers of the primary measuring device provided the manufacturer of the liquid level and flow indicating, recording and totalizing equipment with all the information required for the design and calibration of the associated instrumentation.

Flow Indicating, Recording and Totalizing Instrumentation

The flow indicating, recording and totalizing equipment for each primary measuring device (flume, open-flow nozzle, or gate orifice) was of the head measuring type. The head was measured by the use of a constant flow purge meter assembly using nitrogen gas flowing from a double pressure reducing station mounted on high pressure cylinders. The gas cylinders were stored inside the monitoring station housing.

The flow-head signal was transmitted to a direct reading, flow indicating lenearized in the instrument enclosure and transmitted as a linear, 4 to 20 milliamp DC electronic signal to the electronic strip chart recorder and electronic, 8 digit flow totalizer in the enclosure. Metering range and accessory functions for each device are shown in the following Table 6.

Table 6. MONITORING STATIONS - FLOW INSTRUMENTATION

Monitoring Station	Type of Primary Device	Flow Range	Accessory Functions
Humboldt and Wright	Orifice		
	One Bubbler	0-10.5 cfs	1, 3
Ring and Richards	30" Flume		
	Two Bubblers	0-18.0 cfs	1, 2, 3
Auer and Fratney	20" Nozzle		
	Two Bubblers	0-7.0 cfs	1, 2, 3
Auer and Humboldt	36" Nozzel		
	Three Bubblers	0-31.0 cfs	1, 2, 3
Humboldt and Locust	54" Flume		
	Two Bubblers	0-78.0 cfs	1, 2, 3
Auer and Booth	36" Flume		
	Two Bubblers	0-28.0 cfs	1, 2, 3
	42" Flume		
	One Bubbler	0-38.0 cfs	-----
Locust and Richards	36" Flume		
	Two Bubblers	0-28.0 cfs	2
	24" Nozzle		
	Two Bubblers	0-11.6 cfs	1, 2, 3
Humboldt and Commerce	60" Flume		
	Two Bubblers	0-100 cfs	1, 2, 3, 4
	Orifice		
	One Bubbler	0-8.55 cfs	-----

Accessory Functions:

1-Control Ejector Sampler
2-Control Upstream Bubbler Gas Supply
3-Measuring System Shutdown
4-Control Detention Tank Startup

The flow instrumentation for certain primary devices controlled the operation of ejector type sewage samplers, as indicated in the preceding Table 6. In such cases, the totalizer would retransmit pulses representing preselected flow increments, to an electric, adjustable, predetermining counter. The counter in turn actuated the proper relays to control valves. The above described system resulted in a sample being obtained each time a given quantity of flow passed through the primary measuring device. Control of valves at the sample container caused the sample to be discharged to it. In addition, the sample ejection system was activated at the start of a sampling cycle; that is, when the bubbler valves were opened, and the flow measurement system was started for a given overflow period.

Control of valves at the sampler caused the sampler to refill immediately following ejection of each sample. At the end of each overflow period, after the measuring and sampling system was shut down, a final sample remained in the ejector sampler. Controls for valves at the sample container were therefore provided so that, during the next overflow period, the initial sample discharged to a drain.

At various stations the instrumentation also controlled the operation of upstream liquid level recording equipment as indicated in the preceding Table 6. Where such accessory function was required, the flow instrumentation included controls for opening a solenoid valve on the nitrogen gas supply for the upstream liquid level bubblers when the flow through the primary device reached 95 percent of capacity.

The flow instrumentation also performed the function of shutting down the gas bubblers and flow measurement systems, by closing the gas bubbler solenoid valves when the flow dropped to zero. Measuring systems were started by bell-type pressure switches, as later described.

The zero flow shutoff circuit incorporated an adjustable delay timer. The timer held the solenoid valves open under zero flow conditions, for an adjustable period of 0 - 15 minutes. The delay timer was necessary to eliminate the surge effects which develop due to varying intensities of runoff during rainfall events.

Liquid Level Recording Equipment

Liquid levels measured by all flow measurement devices, and by all upstream liquid level bubblers, were recorded. The flow measurement and upstream liquid level gas bubbler backpressures were piped to individual, electronic, differential pressure level transmitters. The 4 to 20 milliamp DC liquid level signals were transmitted to electronic, strip chart liquid level recorders. The range on all level records was 0 - 10 feet, except that the range on the recorder for liquid level in the 24 inch sewer at Auer and Humboldt was 0 - 2 feet.

As an auxiliary function, the liquid level instrumentation at the Humboldt and Wright and at the Humboldt and Commerce monitoring stations shut down the flow measurement systems when measured head dropped to a predetermined level. This function was necessary due to the continuous dry weather flow at these locations.

Ejector-Type Samplers and Controls

Each ejector-type sampler consisted of a sample chamber located in the sewage flow channel, pinch-type valves at the chamber inlet and outlet connections, flexible preassembled gas and sample tubing connecting the sample chamber to valving and piping within the monitor station housing, solenoid valves to control sampler operation in accordance with signals from flow measurement instrumentation, sample containers and container refrigerator, and all gas and sewage sample piping and controls.

The sample chambers were sized to obtain a 100 milliliter sample. The chambers were designed with an inlet check assembly so that gas under pressure, when applied in accordance with flow signals, would evacuate the sample to the sample container. Chambers were fabricated of polyvinylchloride, with a Teflon seat and nylon ball. Sample entrance openings were 1/8 inch in diameter and the gas inlet and sample outlet lines were 1/2 inch and 1/4 inch diameter, respectively. Each sample chamber and valve assembly was equipped with a chain lifting device to permit its removal from the sewage flow channel through a 12 inch diameter access pipe.

The sample chamber was connected to the monitoring station piping by four flexible hose connections. Two of these lines were for gas to operate the pinch-type valves, the third line conducted high pressure gas to the chamber for evacuation of the sample. The fourth line conducted the sample to the sample container.

The sample containers were rigid, self-supporting polyethylene plastic drums, with a capacity of 10 gallons. The container refrigerators were standard, commercial units.

Vacuum Type Samplers

The station located at Humboldt and Commerce was equipped with a vacuum type, individual sampler. This vacuum actuated, automatic sampler was capable of collecting 24 individual samples at predetermined intervals. Each of the 24 individual samples was at least 250 milliliters in volume when collected at a sampling lift of 15 feet.

The sampler was equipped with a thermostatically controlled refrigeration unit for cooling and storage of the collected samples and with a vacuum pump, both of which were enclosed in the monitoring station housing.

Bell Type Pressure Switch

Each monitoring station included a bell-type pressure switch for activating the flow measurement system gas bubbler, when overflow conditions occurred. The switch operated on the principal of compressed air trapped within a bell by the rising liquid level which mechanically closed a contact within the monitoring station housing. Contact closure opened a solenoid valve on the gas supply to the bubbler for the primary measuring device. The pressure switch would seal in, and would not close the solenoid valve when the pressure switch opened. The shutoff of gas was accomplished by a signal from the flow measurement equipment when the flow rate dropped to zero. To eliminate surge affects, an adjustable time delay was provided in the pressure switch circuit.

The complete assembly was designed to cause closure at the remotely located pressure switch when rising water levels at the bell location resulted in a liquid depth of 1¼ inch above the bottom of the bell.

Accessory Equipment - Typical for all Sewer Monitoring Stations

Each sewer monitoring station included the following accessory equipment:

Ventilation louvers and fan
Thermostatically controlled heating unit
Two 100 watt, incandescent lighting fixtures
Space for two nitrogen cylinders and pressure reducing valves and piping
Each item of flow and liquid level instrumentation was provided with an engraved nameplate describing the function being indicated or recorded, as well as indicating the primary device such as "Flume", "Nozzle", "Upstream Bubbler", etc. All timers and switches were also identified as to system and function.

Combined Sewer Monitoring Station Housing

The monitoring station housings were sized to accommodate all units and facilities described above. The housings had overall dimensions approximately 6'-6" high x 7'-0" long x 3'-0" deep. Figure 13 shows a typical sewer monitoring station housing arrangement. Photographs of a typical combined sewer monitoring station are shown in Figures 14 and 15.

The monitoring station housings were generally located on parkways between sidewalks and curbs. These locations restricted station depth to approximately 3'-0".

The housings were fabricated of twelve gauge steel, shop primed with red lead primer and finished with two coats of green enamel. Doors were provided to permit opening the entire front and rear of each station for easy access to all equipment, valves and piping. The doors are equipped with cylinder locks and all locks were keyed alike.

All monitoring station components (flow and liquid level instrumentation, valves and piping, samplers, refrigerators, etc.) were factory mounted within the housings. All piping was copper tubing, neatly arranged, and marked or tagged to show line function. Unions were provided at all valves and at equipment connections to facilitate removal for maintenance or replacement and all pressure reducing valves on bubbler supply lines were provided with valved by-passes to permit manual blowdown. The power supply to each station was single phase 120/240 volt, 60 cycles. All units were completely factory wired and connected to a circuit distribution panel.

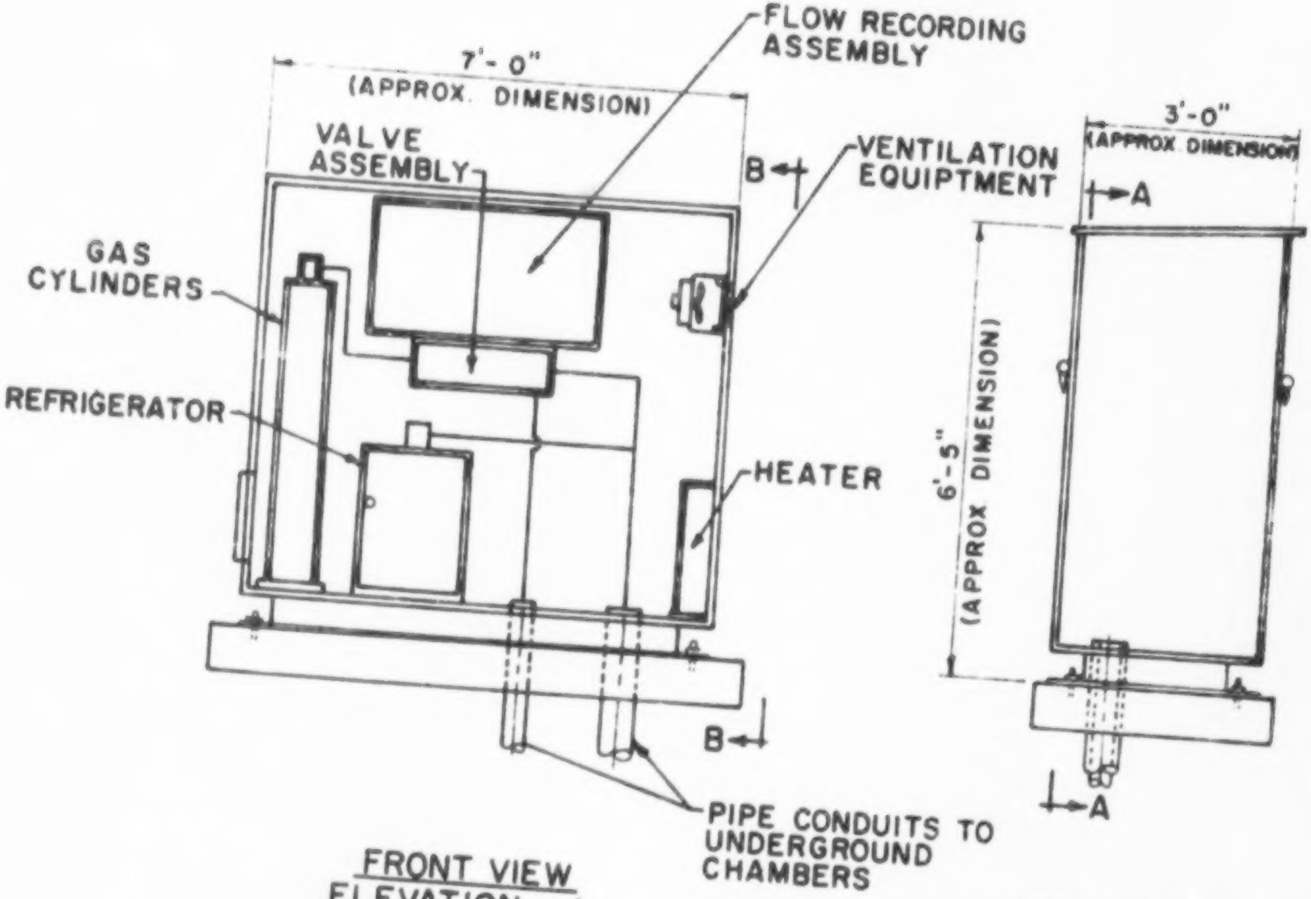
RIVER MONITORING STATION FUNCTION

The function of the three river monitoring stations was to measure, indicate and record the dissolved oxygen concentration and temperature of flow in the Milwaukee River, obtain samples of the river water automatically; and store samples in a refrigerated container.

RIVER MONITORING STATION COMPONENT EQUIPMENT

Vacuum Type Samplers

Each of the three river monitoring stations was equipped with a vacuum actuated, automatic, individual sampler. The samplers were identical to the vacuum sampler previously described for the combined sewer monitoring station at Humboldt and Commerce.


7'- 0"
(APPROX. DIMENSION)
FLOW RECORDING ASSEMBLY
VALVE ASSEMBLY
GAS CYLINDERS
REFRIGERATOR
VENTILATION EQUIPTMENT
HEATER
B
B
PIPE CONDUITS TO UNDERGROUND CHAMBERS
3'-0"
(APPROX. DIMENSION)
A
A
6'-5"
(APPROX. DIMENSION)

FRONT VIEW
ELEVATION A

SIDE VIEW
ELEVATION B

FIGURE 13 - SEWER MONITOR HOUSING AND EQUIPMENT LAYOUT

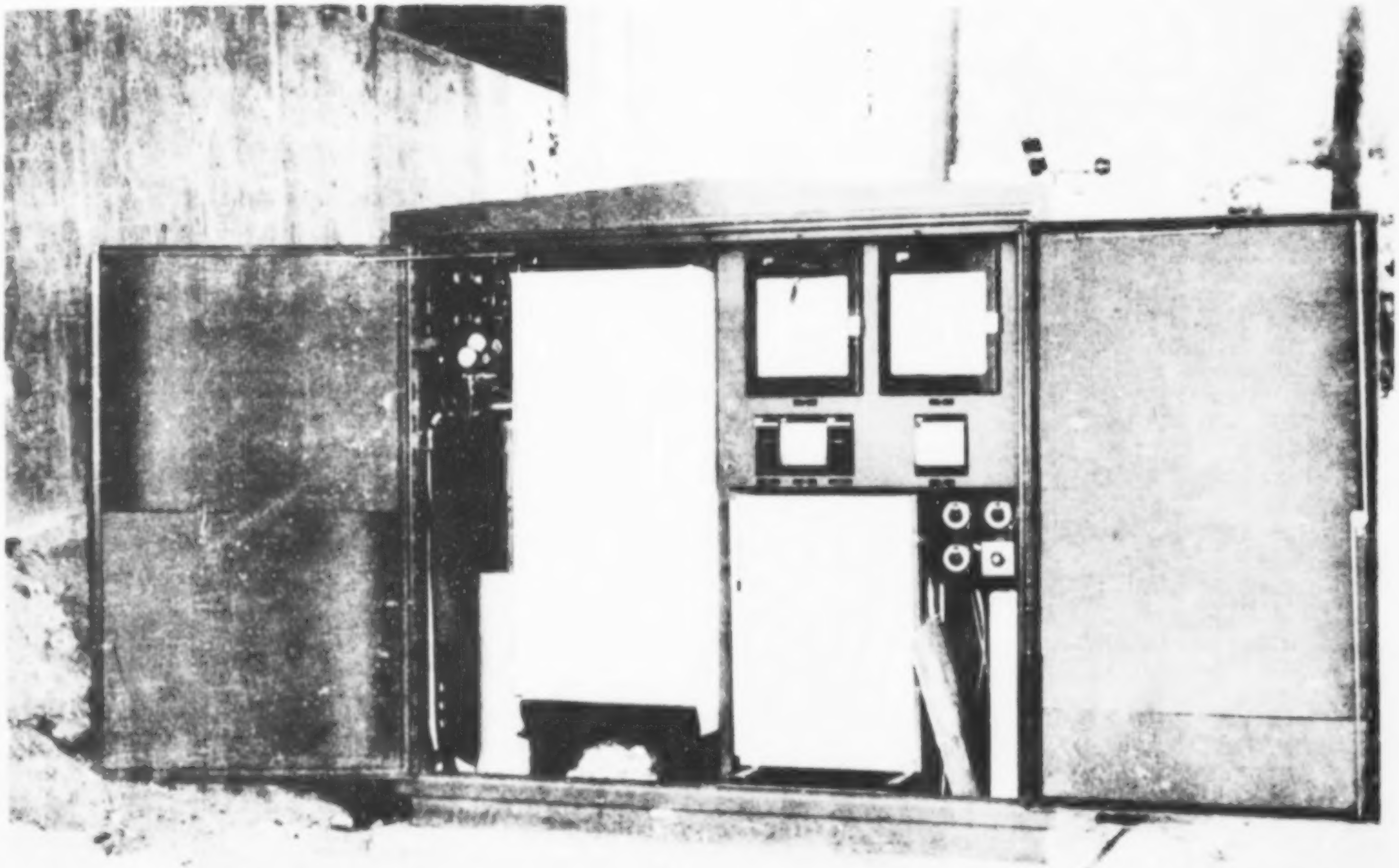


FIGURE 14
CITY OF MILWAUKEE, WISCONSIN
HUMBOLDT AVENUE DEMONSTRATION PROJECT
COMBINED SEWER MONITORING STATION
STATION 49

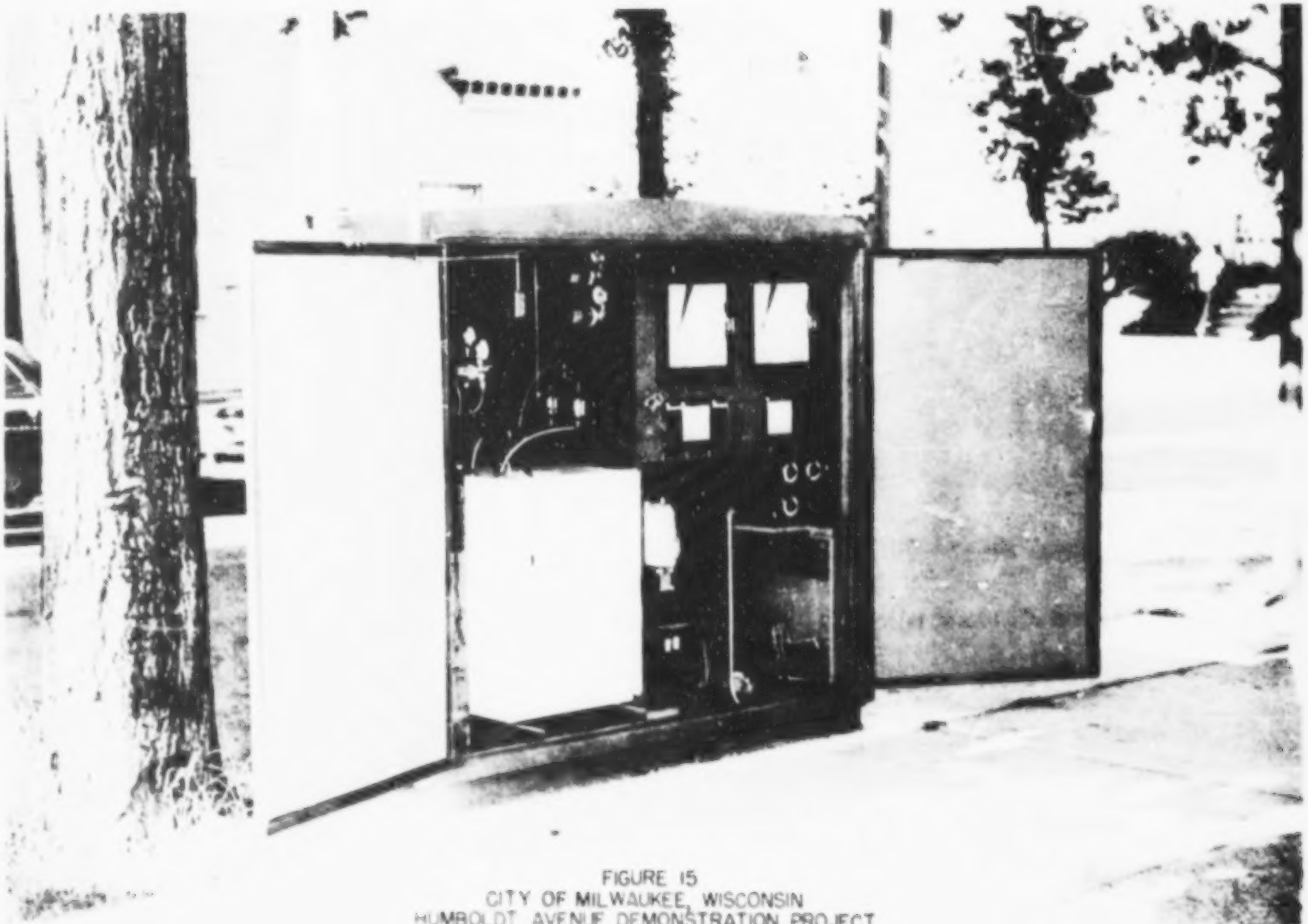


FIGURE 15
CITY OF MILWAUKEE, WISCONSIN
HUMBOLDT AVENUE DEMONSTRATION PROJECT
COMBINED SEWER MONITORING STATION
STATION 41

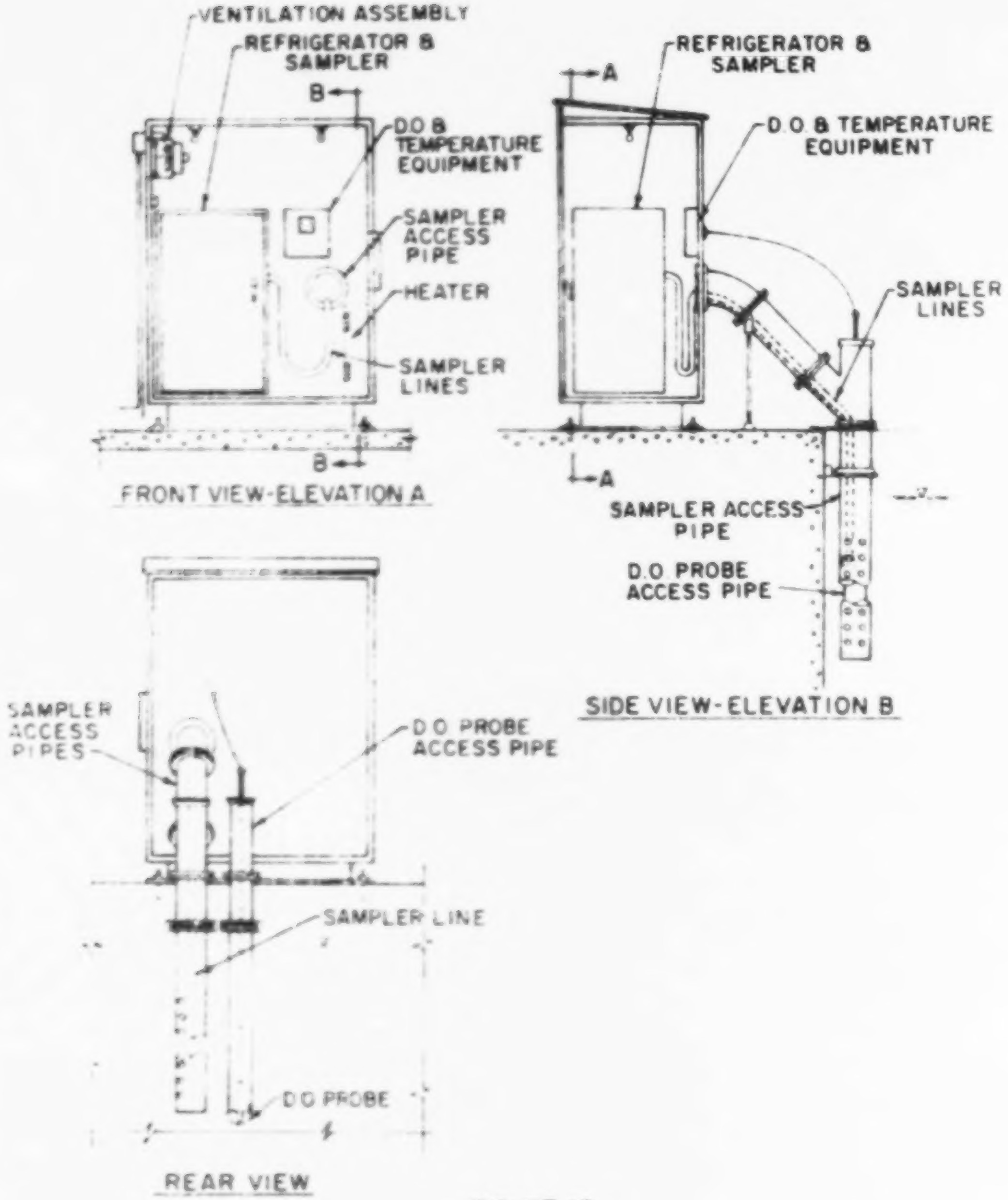

VENTILATION ASSEMBLY
REFRIGERATOR & SAMPLER
B
D.O & TEMPERATURE EQUIPMENT
SAMPLER ACCESS PIPE
HEATER
SAMPLER LINES
B
FRONT VIEW-ELEVATION A
A
REFRIGERATOR & SAMPLER
D.O. & TEMPERATURE EQUIPMENT
SAMPLER LINES
A
SAMPLER ACCESS PIPE
D.O PROBE ACCESS PIPE
SIDE VIEW-ELEVATION B
SAMPLER ACCESS PIPES
D.O PROBE ACCESS PIPE
SAMPLER LINE
D.O PROBE
REAR VIEW

FIGURE 16

RIVER MONITOR HOUSING AND EQUIPMENT LAYOUT

Dissolved Oxygen Concentration Analyzers

The Cherry Street Bridge and St. Paul Avenue Bridge river monitoring Stations were each equipped with one temperature compensated, automatic dissolved oxygen concentration analyzer. To evaluate the effect of the aeration of the river water as it passes over the North Avenue Dam two such devices were provided at this location: one located upstream and one downstream of the dam.

The analyzers were installed to provide an accurate, continuous record of the dissolved oxygen concentrations in the river. However, due to chronic probe malfunction, very little useful data was realized from the use of these probes. The problems encountered in the operation of these analyzers are discussed in Section VII of this Report.

Accessory Equipment - Typical for all River Monitoring Stations

Each river monitoring station also includes the following accessory equipment:

Ventilation louvers and fans
Thermostatically controlled heating unit
Two 100 watt, incandescent lighting fixtures

River Monitoring Station Housing

The river monitoring station housings were very similar to that described for the combined sewer monitoring stations except that they were slightly smaller due to less space requirements. Figure 16 shows a typical river monitoring station housing arrangement. Photographs of a typical river monitoring station are shown in Figures 17 and 18.

RAINFALL GAUGING

In order to ultimately relate U. S. Weather Service records from outside the project study area to the rainfall pattern within the area, a rain gauge installation was required.

During the initial stages of the project, the City of Milwaukee installed a gauge on the roof of the Pierce Elementary School located near the center of the project area. The gauge was a recording, tipping bucket type rain gauge. This gauge was later replaced with a recording, weighing type gauge. Early in the project, vandalism necessitated the relocation of the gauge to another point centrally located in the project area.

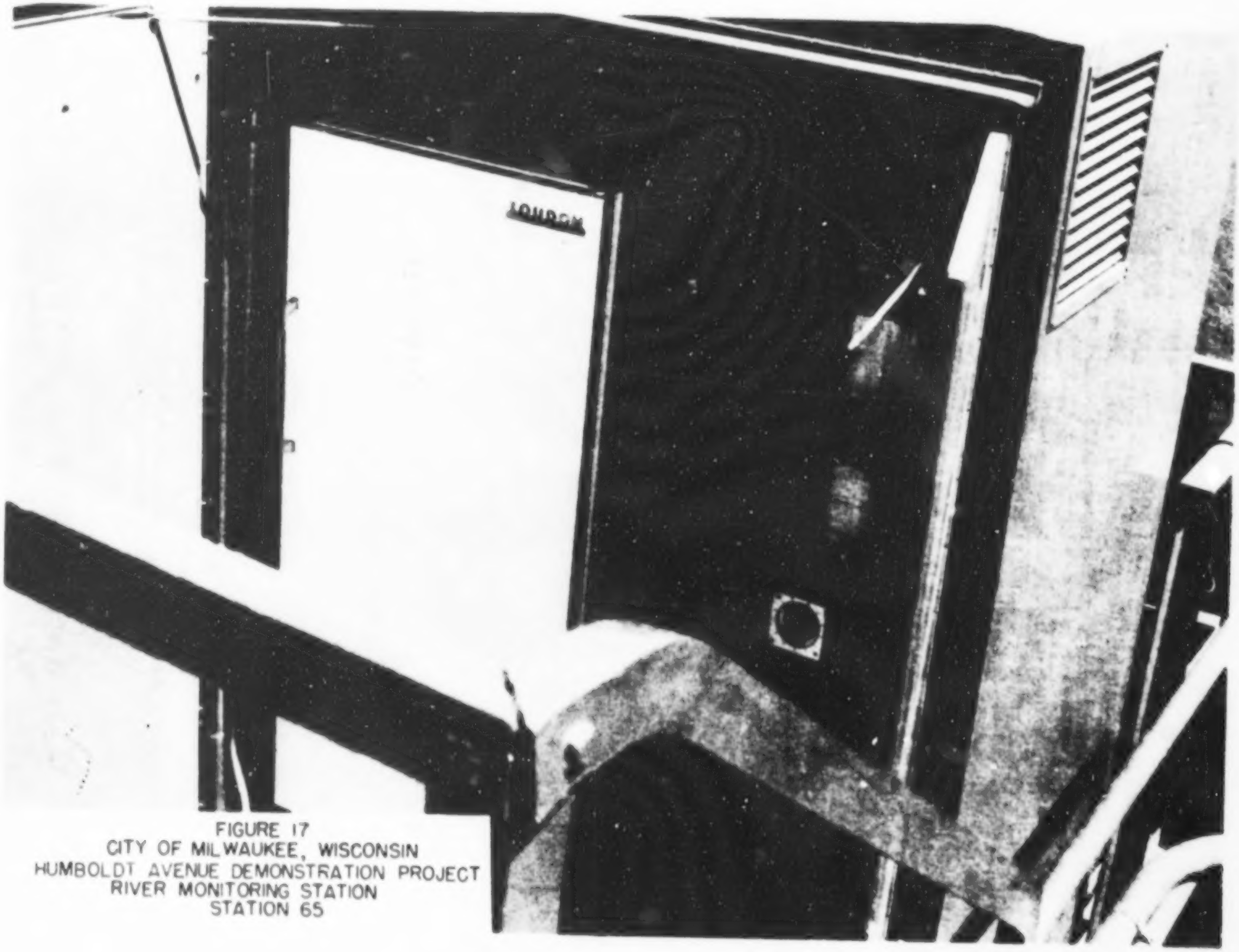


FIGURE 17
CITY OF MILWAUKEE, WISCONSIN
HUMBOLDT AVENUE DEMONSTRATION PROJECT
RIVER MONITORING STATION
STATION 65

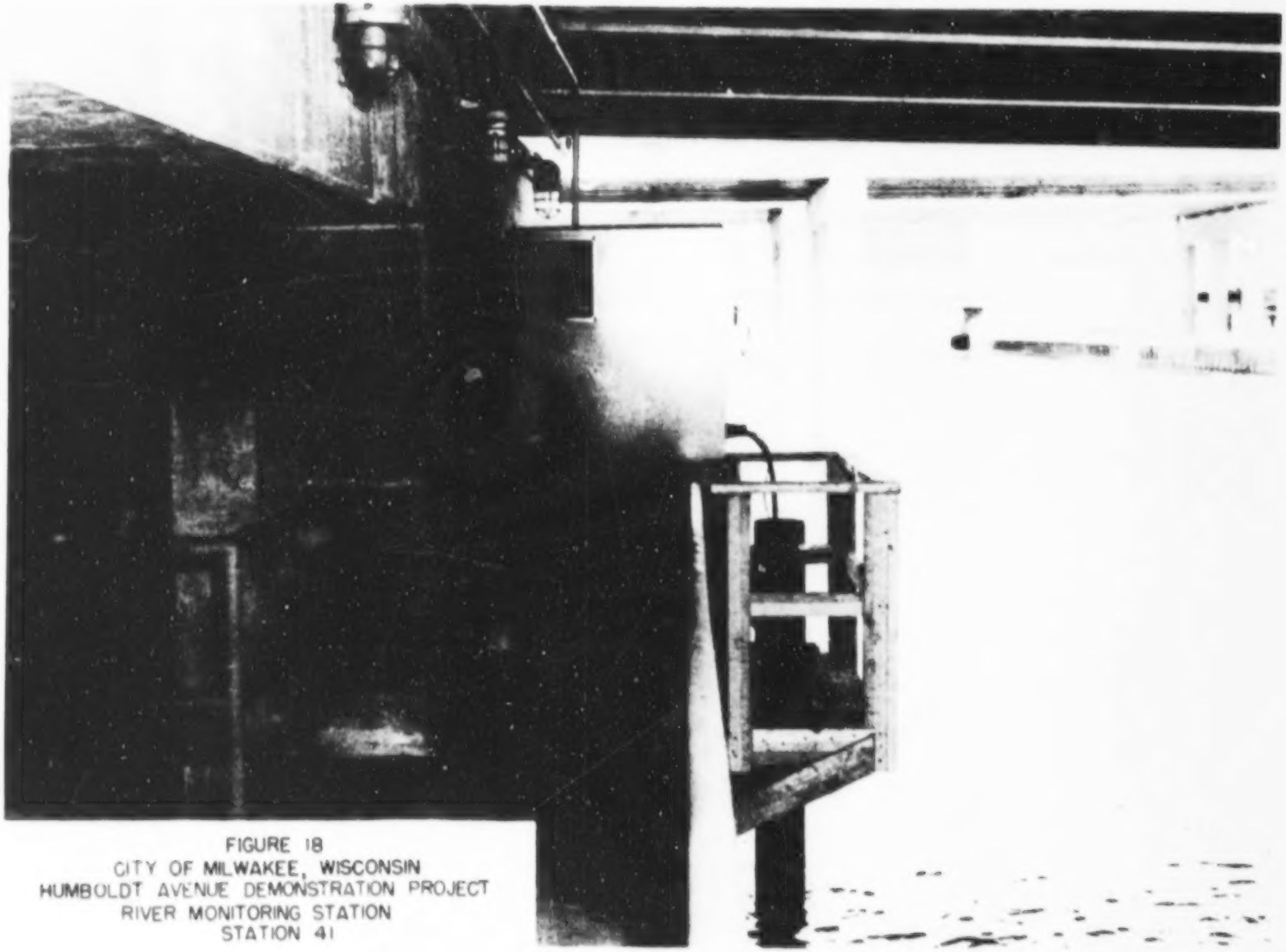


FIGURE 18
CITY OF MILWAKEE, WISCONSIN
HUMBOLDT AVENUE DEMONSTRATION PROJECT
RIVER MONITORING STATION
STATION 41

In addition to the rainfall data obtained from the project area, records from the City recording rain gauge located at the Municipal Building were also collected and studied. This gauge is located directly south of the project study area.

MONITORING STATION EQUIPMENT SUPPLIERS

The various component equipment was supplied by a general contractor in accordance with detailed equipment specifications. The suppliers of the various components are listed below:

Flumes	F. B. Leopold Inc.
Open Flow Nozzles	BIF
Flow Indicating Recording Totalizing, and Ejector Sampling Equipment	Fischer and Porter Co.
Station Fabrication and Auxiliary Equipment	Fischer and Porter Co.
Vaccuum Samplers	Sonford Products
D.O. Concentration Analyzers	Union Carbide Co.

Selection of general contractor and the equipment suppliers was based upon general municipal bidding procedures and does not constitute endorsement or recommendation for use by the City of Milwaukee.

STATION OPERATION AND MAINTENANCE

The operation and maintenance of the monitoring stations was a demanding aspect of this demonstration project. The sampling phase of the project was frequently hampered by equipment malfunction.

To reduce the amount of monitoring station down-time, the City assigned two men to assist the Engineer in the operation and maintenance of the monitoring stations during the final year of data collection. Their presence substantially improved the results of the sampling program. These personnel were able to repair, in the field, many pieces of equipment that the manufacturer's service representative had previously found necessary to remove and return to the factory for repair.

Aside from the equipment operation problems which were encountered and which required additional maintenance time, the routine operation and maintenance of the monitoring stations consisted of the following activities:

1. Replacement of empty nitrogen cylinders.
2. Weekly inspection of piping connections to eliminate nitrogen leaks as they appeared.
3. Monthly change of recorder charts and ink capsules.
4. Cleaning of composite samplers and vacuum sampler lines.
5. Weekly check of the calibration of the level and flow measuring and indicating instrumentation.

To insure that the stations were ready to operate automatically, the following activities were performed immediately after each rainfall:

1. Samples were picked up from the stations and delivered to the laboratory for analysis.
2. Totalizers were checked and total flows recorded for use in preparing samples for laboratory work.
3. Pressure bells were checked and any rags, debris, etc. caught on them by the rising sewer level were removed.
4. Nitrogen bubbler lines were blown out to remove any grease or other substance which may have collected in the open end of the lines.
5. The vacuum sampler at the Humboldt and Commerce monitoring station was reset.

The routine activities necessary to the operation and maintenance of the stations, can be accomplished in one or two days per week with a few hours extra during periods of rainfall necessary to collect samples and perform the above post-rainfall tasks.

A detailed discussion of problems encountered in operation of the monitoring stations, together with recommendations for resolving such difficulties in future studies, is presented in Section VII.

MONITORING STATION COST DATA

Detailed plans and specifications for the monitoring stations were completed early in 1968. Bids were received on March 20, 1968 when the Engineering News-Record Construction Cost Index was 1117.15. The low bid for the eight sewer and three river monitoring stations was $278,818, with changes and alterations to the project increasing the construction cost by approximately $3,000. The total construction cost of $281,818 reflects a per monitoring station cost of approximately $25,600.

The installed prices for the various major items of equipment quoted by the low bidder as well as the estimated cost of each monitoring station is given in Table 7.

Table 7. MONITORING STATION COST DATA

Major Equipment Items	
Monitoring Stations Including All Instrumentation	$ 121,000.00
Flumes	8,930.00
Open Flow Nozzles	7,633.50
Station Cost Including Major Equipment Items	
Combined Sewer Stations	
Station 21 - Humboldt and Wright	$ 19,000.00
Station 25 - Ring and Richards	24,000.00
Station 29 - Auer and Fratney	24,000.00
Station 33 - Auer and Humboldt	25,000.00
Station 37 - Humboldt and Locust	31,000.00
Station 41 - Auer and Booth	38,000.00
Station 45 - Locust and Richards	42,000.00
Station 49 - Humboldt and Commerce	44,818.00
River Stations	
Station 52 - St. Paul Bridge	10,000.00
Station 58 - Cherry Street Bridge	10,000.00
Stations 64 and 66 - North Avenue Dam	11,000.00
TOTAL	$ 278,818.00

Individual Station Costs are based on engineers' estimates - a breakdown of each station's cost was not requested of the Contractor.

SECTION VI

DETENTION TANK

GENERAL

The Humboldt Avenue Combined Sewer Overflow Detention Tank is located at the primary outlet for the demonstration project combined sewer area. The tank was constructed underground on the north bank of the Milwaukee River on North Commerce Street just west of North Humboldt Avenue. Figures 4 and 6 show the detention tank location.

The tank receives combined sewer overflow from a 570 acre study area shown in Figures 2, 3 and 6. Two relief sewers, which traverse the area and the Sewerage Commission's intercepting sewer remove from the system a substantial amount of the total combined sewage generated within the study area before it reaches the detention tank.

The effective capacity of the tank is approximately 3.9 million gallons and the approximate interior dimensions are: 420 feet long, 75 feet wide and 16 feet deep. For design purposes it was estimated that the tank would receive the following maximum inflow rates:

Approximate 5 year storm flow	270 cfs
Approximate 10 year storm flow	320 cfs
Approximate 20 year storm flow	380 cfs

The above estimated detention tank influent flow rates were utilized where applicable in selecting and sizing the various equipment components which were to be incorporated into the project. The maximum values were utilized in developing the hydraulic elements of the detention tank and the combined sewerage conduits.

The detention tank volume was selected based on several factors. The major factors are noted below:

1. Site Capacity - In order to be able to take full advantage of the available site, several alternate tank layouts were investigated. Various depths and horizontal shapes were evaluated. Based upon the site limitations, as well as the

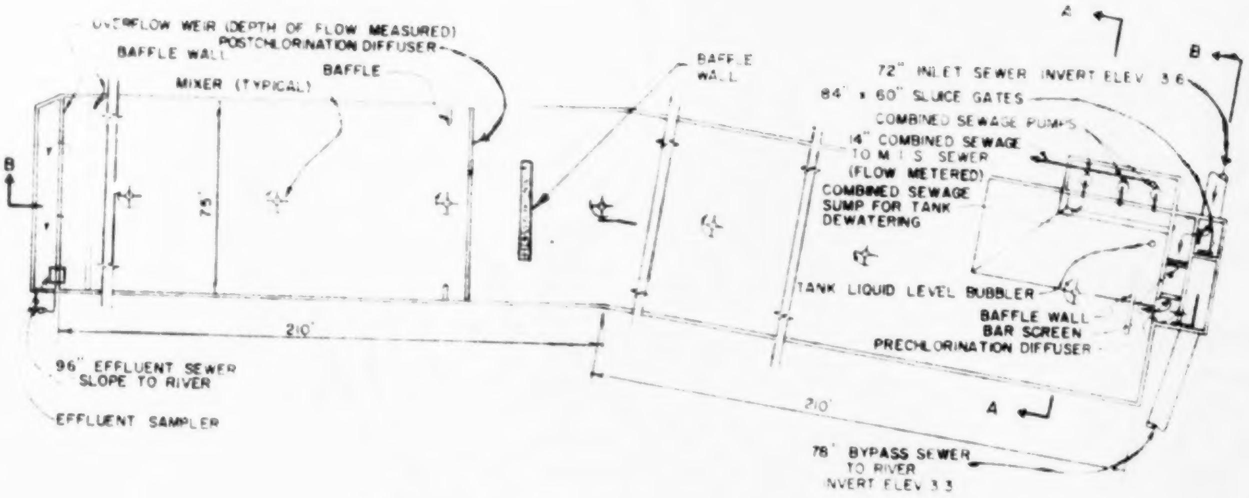
anticipated funds available, it was determined that a tank with approximately 3.9 million gallons of capacity would be the maximum practical size.

2. Volume of Capacity versus Volume of Rainfall Per Storm - A rainfall histogram based on 16 years of record was developed for the project. This histogram is included in Appendix VI. The histogram was utilized to establish the approximate number of rainfall events for which the tank volume would be exceeded and thus cause overflow to the River. It was planned that this number of events, which would exceed the tank capacity, would not be too large as to prevent effective pollution control but still be economically reasonable.

3. Economic Considerations - The project was initially arranged with a specific budget allowance for the detention tank construction. When the detailed design concept of the tank was finalized, it was determined that the 3.9 million gallon capacity tank might exceed the budget available. For this reason and in order to provide the maximum practical tank volume for the budget available, bids were taken on five alternate tank sizes. Bidders quoted prices on a 3.9 million gallon capacity and also on four alternate size tanks of smaller capacity. As can be noted in the portion of this Section on Detention Tank Cost Data, the cost per unit of detention tank volume was significantly less for the largest tank size than for the alternate sizes. This factor was weighed heavily in establishing the tank size.

Due to the size of the tank and its depth, considerable uplift pressure forces were anticipated. During pre-design site investigation, an extremely hard soil condition was found to exist about 20 feet below existing grade which made the use of hold-down piles for resisting the uplift pressures impractical. As a solution to this problem, the final design called for the placement of approximately eight feet of fill over the top of the tank to resist the anticipated uplift. The fill area was landscaped and could be utilized for recreational purposes at some later date.

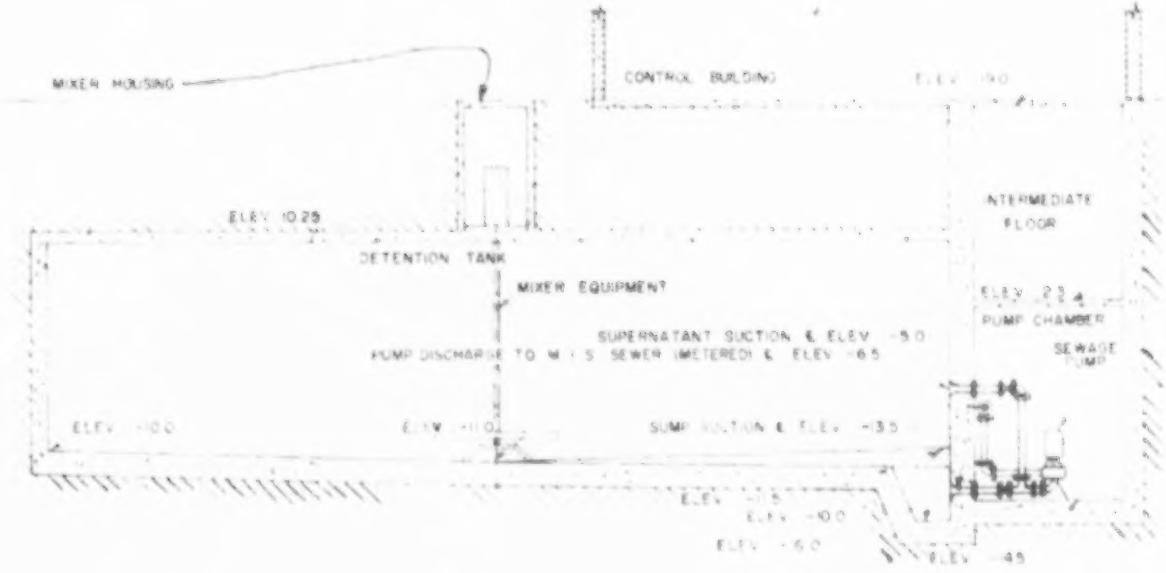
The detention tank and its component equipment are indicated in Figures 19, 20 and 21. A photograph of the tank is included as Figure 22.


A
A
B
B
OVERFLOW WEIR (DEPTH OF FLOW MEASURED)
POSTCHLORINATION DIFFUSER
BAFFLE WALL
BAFFLE
MIXER (TYPICAL)
75'
210'
96" EFFLUENT SEWER SLOPE TO RIVER
EFFLUENT SAMPLER
BAFFLE WALL
72" INLET SEWER INVERT ELEV 3.6
84" x 60" SLUICE GATES
COMBINED SEWAGE PUMPS
14" COMBINED SEWAGE TO M.I.S. SEWER (FLOW METERED)
COMBINED SEWAGE SUMP FOR TANK DEWATERING
TANK LIQUID LEVEL BUBBLER
BAFFLE WALL
BAR SCREEN
PRECHLORINATION DIFFUSER
210'
78" BYPASS SEWER TO RIVER INVERT ELEV 3.3

SECTIONAL PLAN

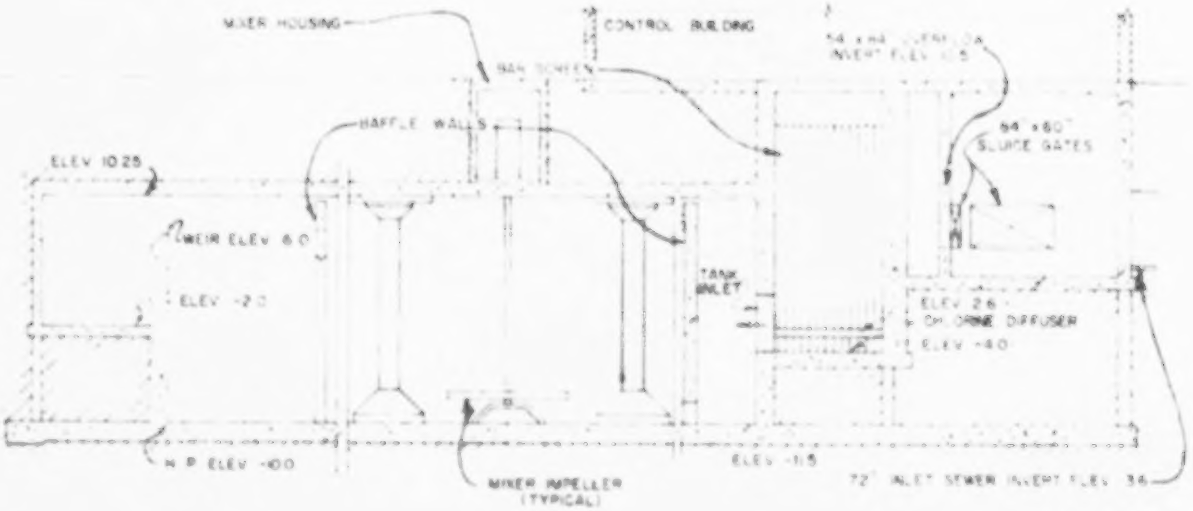
NOTES: INLET FLOW METERED & SAMPLED AT MONITORING STATION 49. SEE FIGURES 20 & 21 FOR SECTIONS

FIGURE 19
CITY OF MILWAUKEE, WISCONSIN
HUMBOLDT AVE. DEMONSTRATION PROJECT
DETENTION TANK-SECTIONAL PLAN


MIXER HOUSING
CONTROL BUILDING
ELEV 19.0
ELEV 10.25
DETENTION TANK
MIXER EQUIPMENT
INTERMEDIATE FLOOR
ELEV 22.4
PUMP CHAMBER
SEWAGE PUMP
SUPERNATANT SUCTION ℄ ELEV -5.0
PUMP DISCHARGE TO M.I.S. SEWER (METERED) ℄ ELEV -6.5
ELEV -10.0
ELEV -11.0
SUMP SUCTION ℄ ELEV -13.5
ELEV -11.5
ELEV -10.0
ELEV -6.0
ELEV -4.5

SECTION A-A

FIGURE 20
CITY OF MILWAUKEE, WISCONSIN
HUMBOLDT AVE. DEMONSTRATION PROJECT
DETENTION TANK-SECTION


MIXER HOUSING
CONTROL BUILDING
54" x 44" OVERFLOW
INVERT ELEV 10.5
BAR SCREEN
BAFFLE WALLS
64" x 60" SLUICE GATES
ELEV 10.25
WEIR ELEV 6.0
ELEV -2.0
TANK INLET
ELEV 2.6
CHLORINE DIFFUSER
ELEV -4.0
H.P. ELEV -10.0
MIXER IMPELLER
(TYPICAL)
ELEV -11.5
72" INLET SEWER INVERT ELEV 3.6

SECTION B-B

FIGURE 21
CITY OF MILWAUKEE, WISCONSIN
HUMBOLDT AVE. DEMONSTRATION PROJECT
DETENTION TANK-SECTION

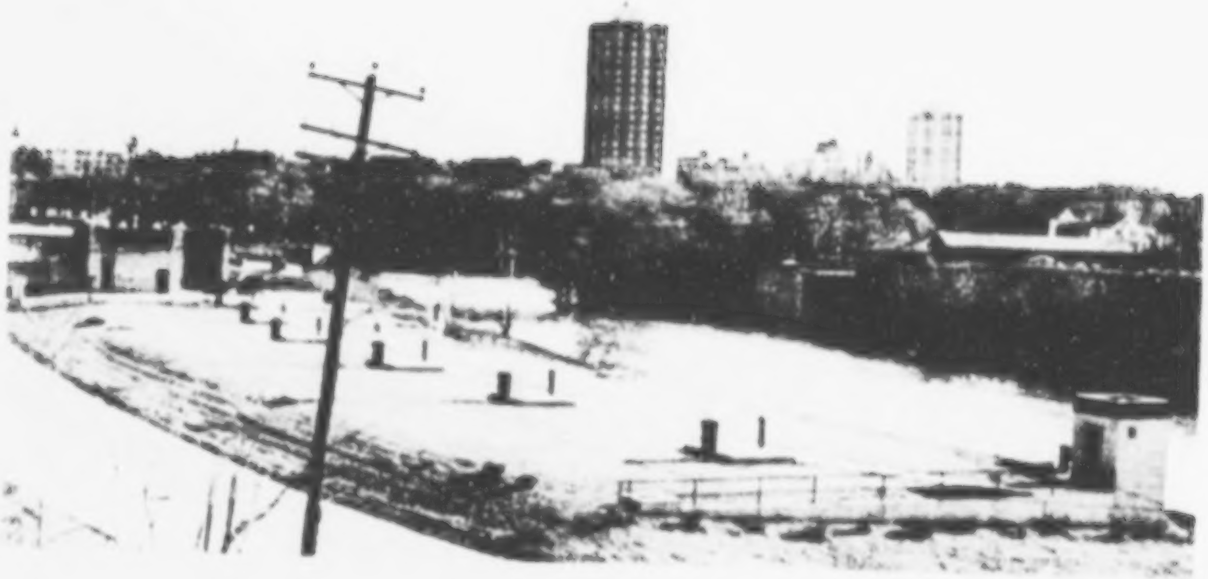


FIGURE 22
CITY OF MILWAUKEE, WISCONSIN
HUMBOLDT AVENUE DEMONSTRATION PROJECT
DETENTION TANK

DETENTION TANK COMPONENT EQUIPMENT

Summary of Equipment Data

Following is a summary of the detention tank component equipment data:

BAR SCREEN

Manufacturer	Rex Chainbelt, Inc.
Number of Units	1
Type	Heavy Duty-Mechanically Cleaned
Clear Openings, Inches	1.5
Capacity, CFS	580

MIXING EQUIPMENT

Manufacturer	Mixing Equipment Co., Inc.
Number of Units	7
Type	Rotary
Motor Horsepower, Each	40

COMBINED SEWAGE PUMPS

Manufacturer	A.G. McKee & Co. (WEMCO)
Number of Units	3
Type	Non-clogging, Vortex Type Vertical, Pedestal Mounted
Pump No. 1	Two Speed
Pump No. 2	Constant Speed
Pump No. 3	Constant Speed

Pump No. 1

Capacity, GPM	400	700
Total Dynamic Head, Feet	17.5	10
Speed, RPM	870	
Capacity, GPM	400	900
Total Dynamic Head, Feet	30	17.5
Speed, RPM	1,170	

Pump No. 2

Capacity, GPM	500	1,500
Total Dynamic Head, Feet	30	10
Speed, RPM	1,170	

Pump No. 3

Capacity, GPM	2,200	3,200
Total Dynamic Head, Feet	30	10
Speed, RPM	870	

SLUICE GATES AND OPERATORS

Manufacturer	Rodney Hunt Company
Number of Units	2
Size, Inches	84 x 60
Type	Rising Stem

CHLORINATION EQUIPMENT

Manufacturer	Wallace & Tiernan Company
Chlorinators:	
Number of Units	2
Capacity (each), Lbs./day	8,000
Chlorine Evaporators:	
Number of Units	2
Capacity (each), Lbs./day	8,000
Chlorine Residual Analyzer:	
Number of Units	1
Chlorine Leak Detector:	
Number of Units	1
Type	Continuous Sampling
Chlorine Flow Recorder:	
Number of Units	2
Type	Differential Pressure Orifice
Metering Range, Lbs./day	0-8000
Indicator Type	Direct Read-Out of Remaining Chlorine
Chlorine Cylinder Hoist:	
Manufacturer	Shepard-Niles
Number of Units	1
Capacity, Tons	2
Type	Geared, Chain Driven

Chlorine Residual Sample Pump:

Manufacturer	Robbins & Meyers, Inc.
Number of Units	1
Type	Horizontal, Helical Screw Positive Displacement
Capacity, GPM	20
Total Dynamic Head, Feet	35

PROCESS MEASUREMENT AND CONTROL EQUIPMENT

Manufacturer	Fischer and Porter
Type	Bubbler

OVERFLOW SAMPLER

Manufacturer	Sonford Products Corporation
Number of Units	1
Type	Automatic Vacuum Actuated
Sample Volume	200 ml at 20'

Selection of the various contractors and the equipment suppliers was based upon general municipal bidding procedures and does not constitute endorsement or recommendation for use by the City of Milwaukee.

Bar Screen

All combined sewage entering the detention tank passes through a mechanically cleaned bar screen located in the tank influent channel. The screen is designed to pass a peak flow of 380 cfs. The purpose of the screen is to remove sewage solids, rags and other debris which otherwise would settle and pile up on the tank bottom or be drawn into the tank dewatering pumps. Screenings are discharged to large portable containers and are then removed to a land disposal site along with all other refuse collected in the City.

Detention Tank Mixing Equipment

The detention tank serves as a settling basin, therefore the deposit of solids over the tank bottom must be removed.

One design concept contemplated the use of bottom scrapers for the removal of settled solids. The final tank layout however made the use of such scrapers impractical. The final design therefore was not based on separate removal of sludge but relied on a system of mixers designed to resuspend the settled solids into the liquid within the tank during the time the tank contents are pumped to the intercepting sewer.

Seven rotary mixers are located within the tank, each driven by a 40 hp motor. Each stainless steel mixer impeller is 7 feet [illegible] inches in diameter and was installed approximately 22 inches above the tank floor. The impellers were set at this elevation, which is adjustable within a 36" range. Studies were not made relating to most efficient mixing impeller elevation or to operation of only a selected number of the mixers during certain periods to improve the solids suspension capability. The mixer drive equipment is located above each mixer on the tank roof, enclosed in a concrete housing which terminates at grade. Each chamber is entered through access manholes.

The mixers were designed to resuspend, within one hour after start-up, all entrapped solids to the following extent:

Solids having settling rate of 1 ft/min or less:

Uniform suspension throughout the basin.

Solids having settling rate of 1 to 4 ft/min:

Off bottom motion.

Solids having settling rate of 4 to 8 ft/min:

Off bottom motion in radius of 30 feet from mixer.

During the first year of tank operation, the mixers satisfactorily performed their intended function with the result that it has not been necessary to manually clean the detention tank after receiving approximately 181 million gallons and detaining approximately 121 million gallons. Solid deposits, consisting mainly of sand and silt, accumulated to a maximum depth of approximately 11 inches and were limited to small areas, three to six feet in width, along the perimeter of the tank. These areas of deposit changed location along the tank perimeter during the year, possibly due to one or more of the following variable factors:

1. Influent flow rate
2. Effluent pumping rate
3. Mixer speed
4. Solids concentration in influent flow
5. Length of detention time

A log of the sediment accumulated in the detention tank as of March 20, 1973 is included in Figure 23.

Tests during future operation of the tank could possibly determine the causes of the deposits and indicate what remedial action is necessary should the continuing deposit of solids be deemed undesirable. To date this is not the case.

One of the seven rotary mixers is equipped with a two-speed motor drive. This mixer, located approximately 150 feet from the tank overflow weir, is operated at low speed prior to and during periods of tank overflow to distribute chlorine for disinfection. It is felt that this slow speed mixer operation may be causing solids loss during overflow, to increase somewhat.

Each mixer is rated to be suitable for operation as a surface aerator. The rating indicates that with a tank liquid depth of approximately 48 inches and 20 percent oxygen available in ambient atmosphere, each mixer is capable of dissolving oxygen at a rate of 10 pounds per hour. Actual detention tank tests to verify this rating were not performed since this aeration feature is not the main design function of the mixers. Studies into the economics and possible treatment benefits of operating the mixers as aerators may be a subject for future tank operation evaluation.

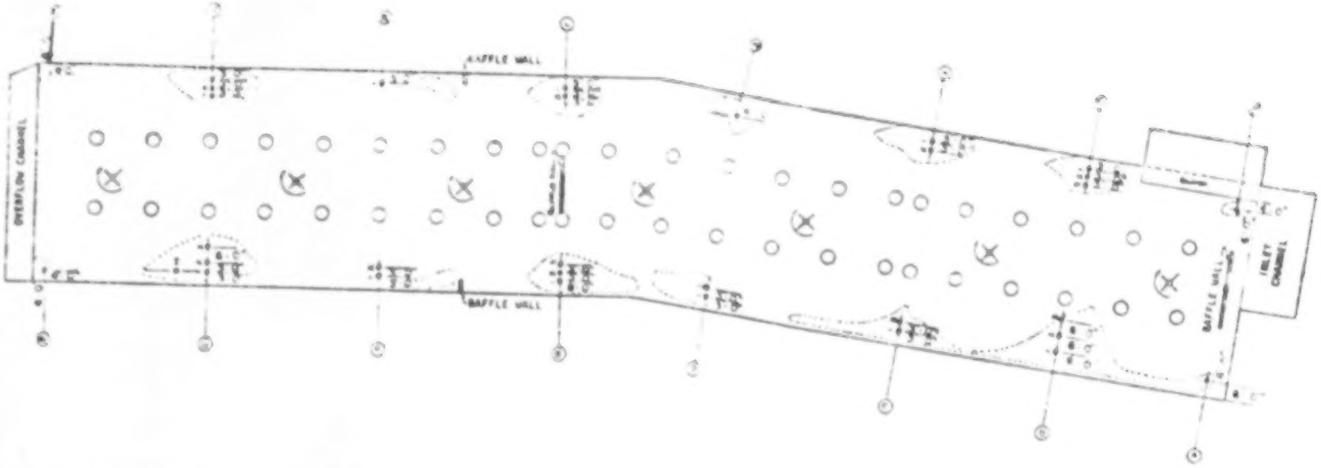
Combined Sewage Pumps

Three vertical, dry pit, pedestal mounted combined sewage pumps are available for dewatering of the tank. Each pump is of the non-clogging, vortex type with recessed impeller suitable for pumping combined sewage.

A sump agitation header was installed in the tank sump to prevent solids build up. Its operation has not proved satisfactory, however it has not been necessary to replace it.

Measurement and Control Systems

All measurement and control systems included in the operation of the detention tank, with the exception of the combined sewage pump discharge measurement system, are of the bubbler tube type utilizing nitrogen or compressed air. The compressed air system is located in the control building while the nitrogen system is located in a monitoring station (Humboldt and Commerce Station) 300 feet from the building. Back pressure, caused by the level being measured, is piped to electronic differential pressure level transmitters, which in turn provide 4-20 ma DC level signals to electronic strip chart recorders.



DATE	A				B				C				D				E				F				G				H				I				J				K				L				M				N				O				P									
	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
10-2-72	2				2	3	6		6	3			6	3			6	4			1				7	5			9				1				6	4			3				1	6			2				7	2			9	7												
3-20-73	2				7	7	4		9	8			7	3			3	2	5		½					3½			14				1				14	9	1		1				5	6	½		7				9	5			1	7			6				14					

NOTES

1 ALL MEASUREMENTS TAKEN FROM INSIDE TANK WALL

2 ENCLOSED AREAS INDICATE APROXIMATE SHAPE OF ACCUMULATED SEDIMENT AS OF LATEST SURVEY (MARCH, 20 1973)

3 DISTANCES NOTED IN TANK ARE HORIZONTAL MEASUREMENTS FROM WALL

4 QUANTITIES IN TABLE ARE INCHES OF ACCUMULATED SEDIMENT

FIG. 23

CITY OF MILWAUKEE, WISCONSIN
HUMBOLDT AVE. DEMONSTRATION PROJECT
DETENTION TANK
LOG OF ACCUMULATED SEDIMENT
(Data and Log provided by City personnel.)

Influent Flow Measurement System - A flume level signal is transmitted via a two-wire transmission line from the sewer monitoring station at Commerce and Humboldt to the detention tank control panel where it is converted to a flow signal and the flow indicated, recorded and totalized. The function of this system is to initiate on-off operation of the bar screen, chlorine evaporators, chlorinators and a protected water pumps. The measured rate of influent flow also controls the rate of prechlorination if prechlorination is being utilized.

Tank Liquid Level System - One bubbler tube is located within the detention tank. The function of this system is to initiate the on-off function of process equipment. At various tank liquid levels this system performs the following operations:

1. Turn off mixers and pumps
2. Change speed of two-speed pump
3. Initiate post chlorination system
4. Initiate tank overflow sampling equipment

Throughout the course of the project the influent flow measurement system was subject to continued malfunctioning. The major problem encountered was the yet unexplained loss of back pressure through the bubbler system. This loss of back pressure occurred at seemingly random moments during periods of sewer overflow and lasted for as long as 15 minutes per occurance during which time the influent flow rate could not be recorded and prechlorination of the tank influent flow on a flow proportional basis was not possible. To provide accurate influent flow rate data, it was necessary to compute the tank volume on a per foot basis such that the increase in tank level could then be converted to rate of flow. This method did provide the required flow data for all storms except those producing runoff in excess of the 3.9 million gallon tank capacity. To obtain the influent flow rate during periods of tank overflow the following measurement system was installed.

Tank Overflow Weir-Head Measurement System - A bubbler tube was installed at the tank overflow weir to measure, indicate, transmit and record the liquid level over the crest of the weir. Using hydraulic formulae the resulting head measurements were converted to rate of tank overflow. A check between this system and that of the influent flow measuring system, when functioning, has proven the substitute system to be as accurate as the system initially intended.

Combined Sewage Pump Discharge System - All combined sewage pumped from the tank passes through a 12 inch magnetic flow meter. The meter transmits a flow proportional, 4 to 20 ma DC signal to indicating, recording and totalizing equipment located at the main control panel.

M. I. S. Sewer Liquid Level System - One bubbler tube was installed in a manhole upstream from the point of tank discharge to measure, indicate, transmit and record the liquid level in the M. I. S. sewer. The system functions to coordinate the operation of the combined sewage discharge pumps with the available level in the M. I. S. thus eliminating the possibility of its surcharge with a resultant undesirable backwater effect.

Tank Overflow Sampler

A vacuum actuated, automatic, individual sampler is located in the sampler building at the southwest corner of the detention tank. The sampler is capable of collecting 24 individual samples at predetermined intervals and is activated whenever the tank approaches an overflow to the river. The sampler head is run into the tank through a 12 inch diameter access pipe to a point approximately 6 feet below the crest of the tank overflow weir. The sampler collects samples approximately 200 milliliters in volume which are stored in a refrigerated compartment until transfer to the laboratory.

Chlorination Facilities

It was concluded that liquid chlorine would be utilized for disinfection of the detention tank overflows and for pre-chlorination to prevent odors. Liquid chlorine was selected as the chlorine source in lieu of various chlorine compounds such as chlorinated lime, sodium hypochlorite or calcium hypochlorite. This selection was based upon the following factors:

1. Cost of liquid chlorine is less
2. Equipment must be capable of an automatic operation. The liquid and gas feeding equipment was best suited for the automated operation.
3. The system must be capable of feeding large and variable quantities of chlorine to the detention tank influent.

The detention tank chlorination facilities include the following items of equipment for handling and feeding chlorine:

2	8000 pounds per 24 hours evaporators with high and low temperature alarms.
2	8000 pounds per 24 hours vacuum operated chlorinators.
4	2000 pound chlorine storage tanks with manifold.
1	amperometric-type chlorine residual analyzer and recorder.

1	portable amperometric titrator used to calibrate the residual analyzer and recorder.
1	chlorine leak detector, to continuously sample the air in the chlorine rooms, and alarm system.
2	chlorine flow recorders to record use of chlorine and indicate remaining chlorine available.

DETENTION TANK OPERATION

Combined sewer overflow is directed to the tank by gravity, through a 78 inch diameter sewer. At the tank inlet the combined sewage enters either the tank inlet channel or a bypass channel depending upon the position of two 84 by 60 inch sluice gates. Use of the bypass channel allows direct discharge of combined sewage to the Milwaukee River, therefore its use is limited to emergency conditions when the tank must be kept empty, which up to the time of this report was not necessary. Upon entering the tank inlet channel, the combined sewage passes through the mechanically cleaned 1$\frac{1}{2}$ inch bar screen. All solid material which is too large to pass through the screen is removed from the sewage flow and deposited in 3 cu. yd. portable disposal containers.

During sewer overflow to the tank, the seven rotary mixers are not operated, except for the low speed operation of the one mixer used to disperse chlorine, and the tank serves as a settling basin. Should a storm be of extreme intensity or long duration and generate an influent flow in excess of tank capacity, this settling results in the removal of a portion of the setteable solids from the combined flow prior to overflow to the Milwaukee River.

After each storm the mixers are activated when pumping commences. The tank contents are thoroughly mixed to resuspend the settled solids so that the pumps deliver the entire tank contents, including the solids, into the M.I.S. for conveyance to the Jones Island wastewater treatment plant.

Chlorination facilities are provided to permit both pre-and post-chlorination of the tank contents. When pre-chlorinating, chlorine is administered in the tank inlet channel on a flow proportional basis. Pre-chlorination is practiced as an aid in odor control. The need for pre-chlorination is subject to many variables including: 1) time of year 2) type and duration of storm, 3) anticipated time to empty detention tank.

The post-chlorination system is activated when tank overflow to the river is anticipated and is administered for the purpose of disinfection.

During post-chlorination, the rate at which chlorine is added is controlled by the chlorine residual as determined by the residual analyzer, as well as on the flow proportional basis.

The pre- and post-chlorination chlorine solution is distributed through diffuser headers. The pre-chlorination header is located in the tank inlet channel. The diffuser runs across the inlet channel just ahead of the inlet parts. The post-chlorination diffuser is located approximately 177 feet from the end of the 420 feet long tank. The diffuser distributes chlorine across the entire 75 feet width of the tank at the point approximately 12 feet above the tank floor.

Piping is also available to administer chlorine solution at the base of each mixer for tank content high rate dosage. To date the effects of utilizing this system have not been studied.

DETENTION TANK MAINTENANCE

During the first year of tank operation, maintenance requirements were carried out by two employees of the City of Milwaukee. The experience of these men and manufacturers' recommendations were utilized in the development of the following list of required maintenance procedures:

Daily

1. Check chlorine residual analyzer acid supply and add acid as needed.
2. Check that chlorine evaporator cathodic protection indicator is in the green range.
3. Replace chlorine leak detector sensitized paper.
4. Check recorder and equipment which is in continuous operation.

Weekly

1. Check operation of sump pump
2. Check air compressor oil level
3. Drain air dryer
4. Visually check ground indicator lights
5. Change chlorine recorder chart and ink pen
6. Renew chlorine residual analyzer buffer solution
7. Check non-continuous operating equipment

Monthly

1. Check seal water lubrication of sewage pumps
2. Inspect insect screen on exterior fresh air intake to boiler room and clean if necessary.

3. Change recorder strip charts and ink capsules on all recorders at main control panel.
4. Clean chlorine residual analyzer sample cell (more frequently with extended use).
5. Operate chlorine residual analyzer sampler and sample pump using manual override (tank liquid level must be at a minimum of 10 feet to take samples).
6. Inspect condition of and operate hand-operated chlorine valves.
7. Operate deluge showers and eyewashers to insure proper operation.
8. Inspect insect screens on vent piping and clean as necessary.
9. Inspect unit ventilator filter and clean as necessary.
10. Check oil level in rotary mixer housing.

Quarterly

1. Grease sewage pumps.
2. Oil heat distributor motor on evaporator No. 1.
3. Grease and lubricate bar screen (more frequently with extended use).
4. Grease rotary mixers.
5. Grease seal water pump.
6. Change oil for air compressor.
7. Purge tank level and M.I.S. level bubbler systems from blow down block located in metering panel.
8. Lubricate chlorine leak detector blower motor bearings.

Semi-Annually

1. Blow down hot water boiler and heater.
2. Check amperage on all 3-phase equipment.
3. Clean unit heater filter.
4. Flush all floor drains to check for blockage.
5. Clean chlorinator rotometer tubes.
6. Clean air-handling unit filter.
7. Grease chlorine residual sample pump.
8. Clean and lubricate trip motor for vacuum sampler.
9. Backwash vacuum sampler tubing (more frequently with extended use).
10. Clean chlorine residual sample pump strainer.
11. Test chlorine leak detector vacuum tubes and replace as necessary.
12. Oil unit ventilator motor.
13. Change air purifier and flow equalizer in chlorine recorder-controller.

Annually

1. Change Oil in mixers.
2. Change Oil in fluid couplings for mixers.

The following maintenance procedures must be performed in addition to those listed above:

1. Change chart on chlorine residual recorder-controller after each use.
2. The portable residual titrator cell must be maintained with distilled water and electrolyte tablets and the cell must be kept submerged in sample or tap water to prevent drying of the cell tip.

PERSONNEL REQUIREMENTS

Prior to tank start-up, it was anticipated that two men would be required to operate and maintain the tank equipment through a three to six month shake-down period. During this time all equipment was to have been checked, adjusted and fine tuned to the point that complete confidence could be placed in the automatic operation of the equipment. Due to equipment malfunctioning, which extended beyond the first six months of operation, and to insure that as much reliable data as possible would be collected, during the one year test period, the City employed two men on a full-time basis. These men were assigned to operate and maintain the tank equipment for the entire final year of the data collection phase of the demonstration project.

It is assumed that normal tank operation, that is considering only those maintenance procedures previously listed, would require the attention of one man for approximately 2 hours per day, seven days per week. During and immediately following a period of rainfall, it would be advisable to have a man visit the tank site to check that all automatic operations functioned properly during the influent period. When it is found necessary to enter the tank or mixer equipment chambers at least two men should be on duty and the proper safety equipment should be employed.

DETENTION TANK COST DATA

Construction Costs

Detailed plans and specifications for construction of the detention tank were completed early in 19[illegible]. Bids were received on July 1[illegible], 19[illegible] and construction began shortly thereafter.

In July, when the bids were received, the Engineering News-Record Construction Cost Index for the Chicago area registered 1,367 and the Environmental Protection Agency Index for the Chicago area registered 135.

Bidders quoted prices on a 3.9 million gallon capacity tank and also on four alternate size tanks of smaller capacity.

The prices quoted by the successful bidder, for each of the five tank sizes, are shown on the following Table.

Table 8. HUMBOLDT AVENUE DETENTION TANK BID PRICES

	Approximate Tank Capacity (Million Gallons)				
	3.9	3.3	2.8	2.2	1.7
Section I General Construction	$1,107,860	$1,045,860	$ 987,860	$ 930,860	$ 870,860
Section II Electrical	39,490	37,546	34,812	31,480	28,580
Section III Plumbing	38,595	38,381	38,167	37,953	37,736
Section IV Heating & Ventilation	26,940	25,789	24,589	23,389	23,620
Totals....	$1,212,885	$1,147,579	$1,085,428	$1,023,682	$ 960,796
Cost Per Gallon of Capacity	$ 0.31	$ 0.35	$ 0.39	$ 0.47	$ 0.57

Costs Include the Cost of the Equipment and Control Building Associated with the Detention Tank.

Construction Cost Projections - Other Facilities

The Humboldt Avenue Detention Tank can provide a basis for estimating costs of similar facilities which may be considered at other locations. In order to utilize costs at other locations, the Humboldt Avenue costs can be related to the ENR Construction Cost Index.

In order to relate the costs to tanks of other sizes, the Humboldt Avenue costs can also be related to unit volumes.

To estimate future tank construction costs, it is necessary to first project the index levels. The following tabulation shows past and predicted index levels for the Chicago area:

Table 9. ENGINEERING NEWS-RECORD CONSTRUCTION COST INDEX
ENR Base: U.S. Average in 1913 = 100 - Labor and Materials

	Index	Increase from Previous Years
December, 1968	1336	
December, 1969	1408	5.3%
December, 1970	1603	13.9%
December, 1971	1837	14.6%
December, 1972, as predicted by ENR	1970	7.3%
December, 1972 Actual	1964	6.9%
December, 1973, as predicted by ENR	2075	5.6%

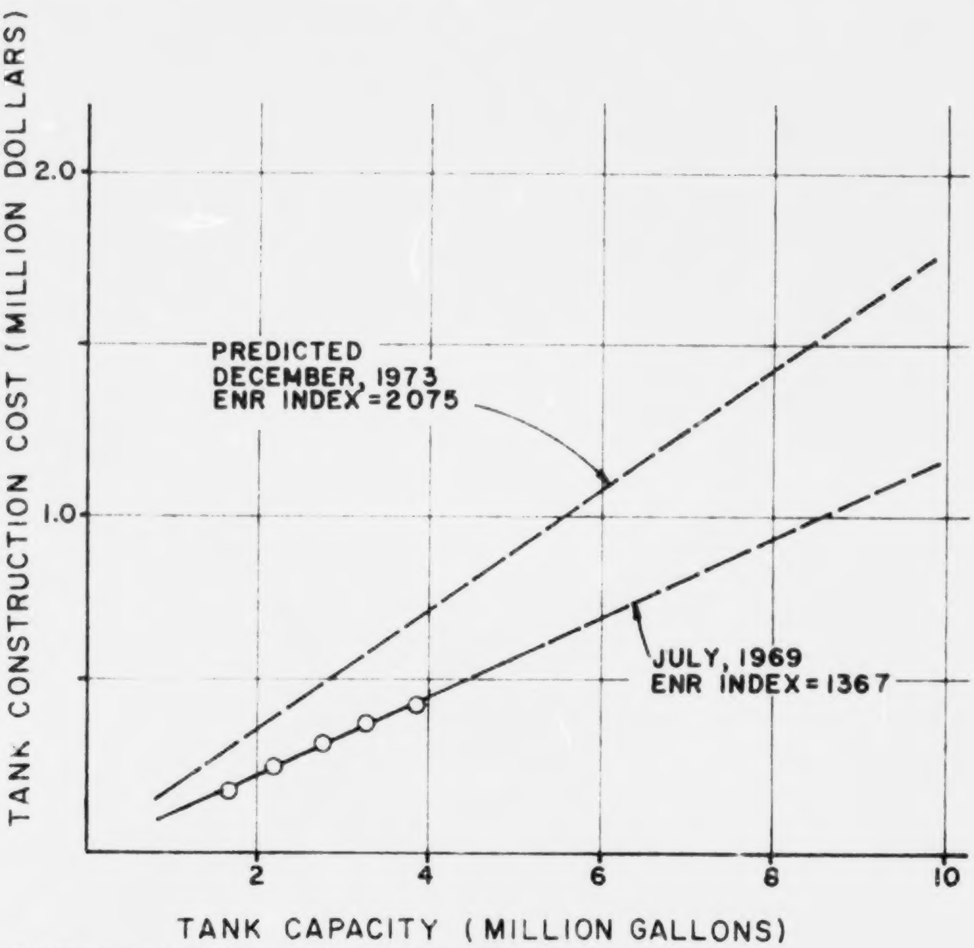
In 1969, the 3.9 million gallon capacity tank cost $1,212,885 to construct. Of this amount, approximately $765,000 was required for construction and equipping of the Control Building and other basic facility appurtenances not specifically related to tank volume. Since a similar building and related equipment would be required regardless of tank size, that cost can be deducted from the total cost to establish the basic tank volume cost. Thus the cost of the tankage only for the Humboldt Avenue Detention Tank was estimated to be approximately $414,000. This cost is meant to be the cost of adding the tank volume to the basic facility of the Control Building, conduits, equipment, etc.

Construction costs, for tanks ranging in capacity from 1.0 to 10.0 million gallons, are shown in Figure 24. Included are costs as existed in 1969 and also costs as predicted for December, 1973 based on the ENR Construction Cost Index. These costs include only the detention tank cost and do not include the cost of the equipment building and appurtenances.

When the data presented herein are utilized as the basis to estimate cost for detention tanks, it may be necessary to incorporate appropriate factors into the estimates for local conditions which would influence cost. Consideration should be given to subsurface conditions,

FIGURE 24

DETENTION TANK CONSTRUCTION COST*
VS
DETENTION TANK CAPACITY



* COST OF TANK ONLY - NOT INCLUDING THE COST OF THE EQUIPMENT BUILDING AND APPURTENANCES

available site size, environment protection restriction, etc. The subsurface conditions of site of the Humboldt Avenue Detention Tank could be classified as relatively difficult construction.

The costs presented in Figure 24 do not include the cost for constructing and equipping the tank Control Building and appurtenances. Figure 25 shows the estimated construction cost for a detention tank building and appurtenances, comparable to the Humboldt Avenue facility, at various ENR index levels.

To estimate total project costs for planning purposes, additional costs must be added to the construction costs presented in Figures 24 and 25. These additional costs can be estimated on the basis of percentage of total construction cost, for the purposes of this Report, as follows:

Engineering (Design, General Inspection and Detailed Inspection)	12%
Construction Contingencies	5%
Fiscal, Legal and Administrative	8%
Miscellaneous Special Studies (Soil Borings, Operation and Maintenance Manual, Engineering Reports in Support of Financing. Environmental Impact Study, etc.)	5%
Total Incidental Cost Factor................	30%

The site of the Humboldt Avenue Detention Tank was acquired by the City prior to the inception of this project and, therefore, an additional cost for land was not a consideration. Land costs being so variable from one location to another, this addition to the total project cost must be separately evaluated for the particular area of study.

Operating Costs

The following costs of operation are based on a one year period of operation between November, 1971 through October, 1972, inclusive. The operating costs presented include costs associated with the operation and maintenance of the Humboldt and Commerce monitoring station:

FIGURE 25
CONTROL BUILDING & APPURTENANCES COST
VS
ENGINEERING NEWS-RECORD CONSTRUCTION
COST INDEX

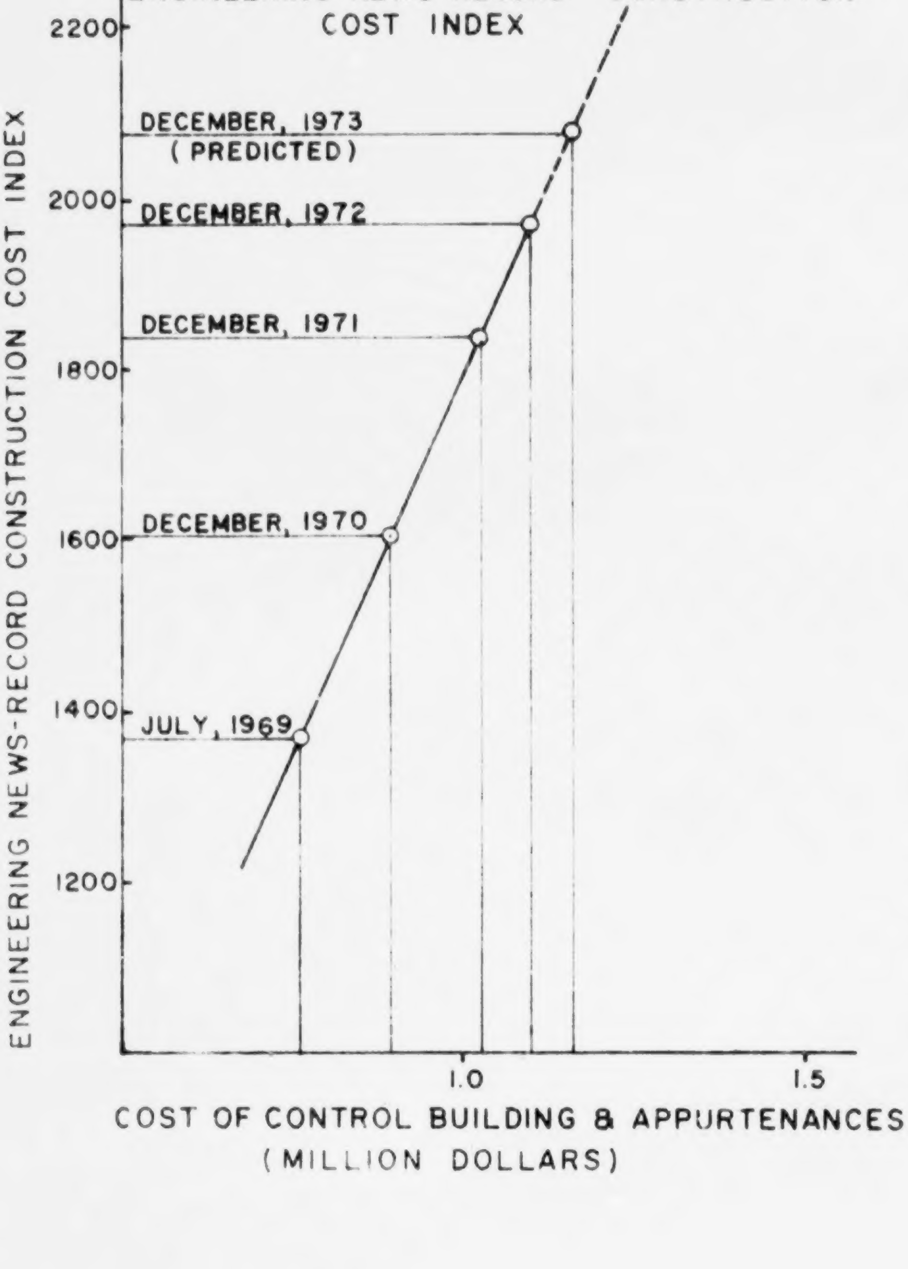


Table 10. HUMBOLDT AVENUE DETENTION TANK OPERATING COSTS

Replacement Parts and Equipment	$ 2,172.51
Nitrogen	94.08
Chlorine	698.00
Miscellaneous Supplies	1,154.47
Utilities (Light, Heat, Telephone, Etc.)	10,059.92
Labor	35,717.35
Total Operating Costs..................	$49,896.33

The cost of labor included in the above Table represents approximately 2.2 man-years. This relatively large amount of labor time resulted from start-up problems during the first year of operation, and from the program of having personnel on intensive duty to insure that as much reliable data as possible would be collected for incorporation into the demonstration project studies. To perform the routine operational and maintenance procedures required at the tank, it is estimated that as little as 0.50 man-years per twelve-month period would be required.

Possible incorporation of a connection to the detention tank compressed air system or a separate system at the Commerce and Humboldt monitoring station would eliminate the cost of nitrogen from the tank operation. The cost associated with replacement parts and equipment was, perhaps, 50% higher than would be expected for first-year operation.

A reasonable estimate of operating cost for a typical detention tank at another location is difficult to develop because costs will vary with tank size, amount of flow handled per year, policy on requirements for 24-hour per day attention at the facility, and other variables. However, as a guide, the following estimate is presented for general budget purposes:

Table 11. GENERAL OPERATING COST BUDGET - TYPICAL DETENTION TANK

Based on 1972 Costs for Labor and Services

Parts and Equipment	$ 2,500.00
Chlorine and Nitrogen	1,500.00
Miscellaneous Supplies	2,000.00
Power and Other Utilities	12,000.00
Labor	12,000.00
Total Estimated Annual Operating Cost Budget.........................	$30,000.00

SECTION VII

DATA COLLECTION PROGRAM

GENERAL

Early in the Demonstration Project planning stage an extensive monitoring program was established. Its purpose was to facilitate the orderly collection of the data necessary in evaluating the effectiveness of the detention tank approach in reducing pollutional loadings to the Milwaukee River caused by overflow from the project study area combined sewer system. The data to be collected was to be used to characterize the effect of the resultant reduction of pollutants on the quality of water in the river and to enable the prediction of the effect of a system of such tanks on the river water quality. In addition, the data was to be used to aid in projecting the suitability of this method of handling combined sewer overflow to areas of varying size and runoff characteristics as is discussed in Section X of this Report.

To satisfy these objectives, it was deemed necessary to thoroughly characterize the following conditions which are major influences on quality and quantity determination:

Sewer System

Dry weather flow and quality:

1. Seasonal variation.
2. Average daily variation during each of the four seasons.
3. Four to six hour variation.
4. Influence of time since, and duration of, last rain storm.

Wet weather flow and quality:

1. Average quantity and quality of overflow as influenced by:
    a. Time of day storm occurs.
    b. Time since last storm.
    c. Intensity and duration of the storm.

2. Variation in overflow quantity and quality during a specific storm as influenced by:
    a. Time of day storm occurs.
    b. Time since last storm.
    c. Intensity and duration of the storm.

Milwaukee River

Dry weather quality:

1. Seasonal variation.
2. Variation during the day in each of the four seasons.
3. Influence of the flushing tunnel.

Wet weather quality:

1. Seasonal variation.
2. Variation during the rainfall event.
3. Influence of the flushing tunnel.

The data collection phase of the project with brief interruptions extended over a five year period, beginning in July, 1967 and continuing through November of 1972. By the nature of its purpose, it was viewed as consisting of two stages.

The first, or pre-tank, stage encompassed all data collected prior to the commencement of detention tank operation. It began with the collection of baseline river water quality and flow information from the Milwaukee River and the river flushing tunnel, which served as background data for the monitoring program. Also included in this stage was the collection of combined sewer wet and dry weather sewage quality and quantity data, Lake intake water quality data, and project area rainfall gauging. Finally in the first stage, other available data pertinent to this project was compiled for review. This included: past rainfall records from the National Weather Service recording station at General Mitchell Field; Milwaukee River discharge data recorded by the U. S. Department of Interior Geological Survey at its Estabrook Park gauging station; and river water quality data compiled by the Sewerage Commission of the City of Milwaukee since 1946.

The second, or post-tank, stage of the monitoring program consisted of the sampling necessary to evaluate the detention tank performance as a treatment unit and was initiated following tank start-up. Included in this stage was the collection of river water quality and flow data; detention tank influent and overflow quality and quantity data; and continuation of wet and dry weather combined sewage quality and quantity data and rainfall data collection.

Following is a discussion of the specific monitoring programs conducted during the five years of data collection. Much of the data compiled during that time is presented in tabular or graphic form in Section VIII of this Report and discussions regarding the ultimate use of the data are presented in Section IX.

RAINFALL DATA

Hourly precipitation data of record, from September of 1948 through November of 1972, for the Milwaukee area was developed. The recorded data was obtained from the U. S. Weather Service which maintains a recording gauge at General Mitchell Field. This location is approximately 8 miles south of the project study area.

Rainfall data are also recorded at two City maintained rain gauge installations. One gauge is located within the boundaries of the project study area and the other is located atop the Municipal Building, approximately 2 miles south of the project area.

Rainfalls were classified by intensity, to one-hundredth of an inch, and by duration for 0 to 10 hours.

Computer processing of the data provided the following information:

1. Printout (with punch cards optional) indicating each event, its corresponding average intensity and duration time, and the time interval to the next event.

2. Each event was classified based on its duration and intensity. It was then added to a frequency matrix for that month of the year, for that month of all years, and for all months of all the years.

3. Printout (with punch cards optional) indicating the frequency matrix of the individual months over all the years and the frequency matrix for all the months over all the years.

SEWER MONITORING

Wet Weather Sewer Monitoring

The eight combined sewer monitoring stations were placed in service in the summer of 1969. However, during the station testing and trial operation period and continuing through 1971, only a limited number of storms resulting in combined sewer overflow in the project area were experienced. This small number of high intensity, long duration storms provided limited opportunities to obtain the data required to characterize wet weather combined sewage.

Further hindering the study were the numerous equipment malfunctions encountered throughout the program. Initial installation and operation of the monitoring equipment appeared to be quite satisfactory, but frequent malfunctioning began to occur shortly after commencement of station operation. The problems mainly centered around the flow measurement systems and their electrical and mechanical controls in the monitoring stations.

The abnormally dry weather conditions and station down-time due to frequent equipment failures combined to result in a minimal amount of wet weather sewer flow and quality data during this period. Fortunately, in the final year of data collection (1971-72) rainfall in the project study area was in excess of average amount of record. Also during the final year, an increase in monitoring station operational personnel decreased the occurances of equipment failure. This, combined with the above average rainfall, resulted in a much improved sampling program.

Sufficient wet weather combined sewage data was gathered during this period of the program to fully characterize the quality and quantity of the flow generated within the project study area. Composite samples were collected at the seven upstream stations on a flow proportional basis. The Commerce and Humboldt monitoring station, located immediately upstream from the detention tank, measured and recorded the flow rate and took samples of all sewer overflows discharging to the tank. Flow proportional composite samples were collected at this station and, in addition, a sampler was provided which permitted the collection of individual samples at predetermined time intervals. The timing device of this individual sampler was normally set to collect samples at 10 to 30 minute intervals.

Dry Weather Sewer Monitoring

Dry weather quality sewer monitoring was restricted to the Humboldt and Wright and Humboldt and Commerce monitoring stations. It was observed that the strengths of the dry weather sewage flow incident to these stations is characteristic of flow generated during dry weather within the entire project area.

The four seasons of the year were represented in the sampling program and many of the samples collected were analyzed on a per hour basis in order to render the data as flexible in use as possible. Samples collected at the Humboldt and Wright station were composite samples collected over 24-hour periods. At the Humboldt and Commerce station, however, the sampling equipment available allowed samples to be collected on either an hourly or composite basis.

DETENTION TANK MONITORING

During periods of tank overflow the detention tank serves as a treatment unit. To determine the degree of treatment being administered to the combined sewage, influent flow characteristics were compared to the characteristics of samples collected at the tank overflow weir. Overflow samples are obtained using an individual sampler similar to the one located at the Commerce Street monitor station. Samples were automatically collected during tank overflows at from 10 to 60 minute intervals.

Flow measurement systems utilized at the detention tank were described in Section VI.

RIVER MONITORING

During the design, construction and initial operation of the monitoring stations, base line water quality and flow information was collected on the Milwaukee River and the river flushing tunnel. This data was required to provide an overall profile of the river as it encounters the combined sewer area of Milwaukee and as it existed prior to the operation of the detention tank.

An important feature of the Milwaukee River, downstream of the North Avenue Dam, is the flushing tunnel which is used to augment the river flow. Samples of the lake water at the tunnel intake were collected on a weekly basis for a total of approximately 8 months during which time the tunnel was in operation. This sampling was required to provide the quality data necessary to characterize the effect of flushing on the river water quality. The full impact of flushing could not be characterized, however, without first determining the rate of discharge through the tunnel.

For this determination, Rhodamine B dye was used to measure the time of travel between the tunnel intake at Lake Michigan and its outlet at the river. The Milwaukee Sewerage Commission provided radio-equipped service vehicles to signal the exact time of dye introduction into the tunnel intake. Samples were taken from a boat stationed on the river approximately 100 feet downstream from the tunnel outlet. Two tests were run for verification using different dye concentrations.

During the first test, samples were collected at one minute intervals and at 15 second intervals during a part of the second tests. The samples were returned to the laboratory and fluorometrically analyzed.

In addition to the sampling, visual observations were made directly at the tunnel outlet. In the first test, using 12 gm of Rhodamine B dye diluted to 1 liter, visual observation of the dye occured 11.5 minutes after its introduction at the tunnel intake and sample analysis indicated a travel time of 12 to 13 minutes from the intake to the point of sampling.

In the second test, 30 gm of Rhodamine B dye diluted to 2 liters was used. Again, visual observation indicated a travel time of 11.5 minutes from the tunnel intake to outlet and laboratory analysis of the samples collected indicated a travel time of 12.75 minutes to the sampling point.

Therefore, using a travel time of 11.5 minutes for discharge calculations, the tunnel discharge rate was found to be approximately 200,000 gallons per minute or 422 cubic feet per second. When this tunnel discharge rate is compared to the normal range of river discharge, as recorded by the U. S. Geological Survey, the importance of the tunnel operation to river water quality becomes quite apparent. The tunnel discharge can account for from 3 to 100 percent of the river flow.

In addition to the flushing tunnel studies, extensive river water quality background data was collected during a period of approximately 21 months beginning in July of 1967. Samples were taken at various locations along the river, generally on a five or six day per week basis. This sampling schedule was interrupted only by severe weather conditions or thick ice cover on the river which prevented sampling.

During this period of background data collection, one intensive 48 hour sampling survey was also conducted. Samples were taken manually at two hour intervals at four locations along the river and at the river flushing tunnel Lake Michigan intake.

To supplement the extensive background sampling, conducted prior to monitoring station construction, additional wet and dry weather river samples were taken using the automatic sampling devices available at the monitoring stations. These samplers are individual, vacuum-type samplers which permit sampling at predetermined time intervals.

River monitoring station design called for the installation of four temperature compensated, automatic dissolved oxygen concentration analyzers. One analyzer was installed at the Cherry Street station, one at the St. Paul Avenue Station and two at the North Avenue Dam monitoring station. Installation of two probes at the dam, one upstream and one downstream was deemed to be desirable to evaluate the effect of aeration above and below the dam.

The four probes originally installed were plagued by numerous mechanical and electrical problems. The probe manufacturer attempted, on several occasions, to remedy the problems encountered but due to continuous malfunctioning the four probes were eventually abandoned - having provided a limited amount of useful data. Their failure did prove that the state of the art in the case of the Milwaukee River, is not yet developed to the degree necessary for automatic operation without nearly continuous maintenance.

In the spring of 1972, two dissolved oxygen analyzers of different manufacture, were installed at the Cherry Street Bridge river monitor station for the purpose of testing and evaluating their performance. For comparison purposes, river samples were manually collected, were analyzed for dissolved oxygen content using the Winkler titration method and the results were compared to the output of each probe. During the course of their evaluation, both probes exhibited an encouraging degree of reliability and accuracy when properly calibrated and diligently cleaned.

It was concluded from the evaluation that the advantage of employing automatic dissolved oxygen analyzers, in lieu of manual collection of samples, is questionable when considering collection of D. O. data on the Milwaukee River. The reasons for this is that the Milwaukee River presents a seemingly unique scum growth problem. During the warm weather months, when D. O. data is of highest importance, any material coming in contact with the river water is soon covered with a coating of an algae-grease substance. As this scum adheres and increases in thickness over the tip of the D. O. probe the output of the probe is gradually diminished.

To preserve the accuracy of the probe output, the probe must be cleaned at least once each day with a dilute solution of hydrochloric acid. This frequent cleaning results in a more rapid change in the probe calibration and, therefore the probes must be recalibrated approximately once each week. To insure that accurate data is being recorded by the probes, samples must be collected manually on a daily basis to determine when recalibration is necessary. The time spent in cleaning and checking the accuracy of the probe could and was better utilized to collect samples for Winkler analysis.

A continuous record of dissolved oxygen was desirable, initially, to assist in the evaluation of the effect of combined sewage overflow on the river dissolved oxygen levels on a random basis. That is, having a continuous record of the D. O. concentrations would have permitted a comparison of D. O. concentration changes to combined sewage overflow quantities and strengths for any storm of record. In not having a continuous recording, it was necessary to collect river samples manually during a storm and then only the effect of that particular storm could be evaluated. A continuous record of D.O. would also be valuable in showing the effect of the flushing tunnel operation on the D.O. in the river.

To obtain sufficient data to satisfy the requirements of the demonstration project, it was necessary to conduct four intensive, dissolved oxygen sampling surveys. Two of the surveys were conducted during periods of dry weathe , to identify the base line D. O. conditions in the river, and two surveys were conducted during periods of wet weather to determine the effect of combined sewage overflow on the D. O. levels.

The four surveys were scheduled such that a wet and a dry weather survey were conducted prior to initial operation of the detention tank and the final two surveys were made after the tank had been placed into service.

The first two surveys were five days in length and the final two surveys were extended to eight days in length. During the surveys, samples were collected at various locations along the river at 2 to 4 hour intervals and at least once per day at various spatial locations. The dissolved oxygen concentration of each sample collected was determined by the Winkler titration method.

The Milwaukee River locations and the spatial locations at which samples were collected during these surveys included the following:

	Station Number	Identification
Milwaukee River	52	St. Paul Avenue
	58	Cherry Street
	59	South Water Street
	62	Humboldt Avenue
	66	North Avenue Dam - Upstream
	65	North Avenue Bridge
Spatial Locations	40	Flushing Tunnel Intake
	47	Milwaukee River Harbor
	81	Menomonee River at South 2nd Street
	82	Kinnickinnic River at Kinnickinnic Avenue

LABORATORY ANALYSES

All samples collected were delivered to the Marquette University Sanitary Engineering Laboratory for analysis. All chemicals and laboratory equipment necessary for the intended analyses were readily available at the lab.

Laboratory analyses of the samples were conducted under the direction of Dr. Raymond J. Kipp. At all times during the demonstration project, highly qualified technical personnel were employed to expedite sample analyses. A qualified technician was employed on a full time basis for the most of the five years of the project and part-time assistance was provided by senior students in environmental engineering or graduate students from the same area.

The parameters chosen for investigation during the project were classified into groups of descending importance. The degree of importance of each parameter was based on the relative utility of each in analyzing and modeling the performance of the storm tank in one case, and the river impact in the other. Table 12, following, summarizes the analyses which were performed during the program. Table 13 indicates the order of importance of each parameter for each system.

The following schedule of analyses, presented in Table 13 was divided into three parts to indicate the order of data preference and is interpreted as follows:

(A) To be determined in all cases-data is critical to analysis of tank performance and river response.

(B) Data is not critical to development or evaluation of system models. Data is important for developing a comprehensive picture of system quality, but occasional gaps in data will not be detrimental.

(C) Data is of limited value to system analysis or shown by prior data to be of low concentrations and/or small variations in concentration.

The analytical results from combined sewage and tank overflow samples were used to characterize performance of the detention tank as a treatment unit. BOD, suspended solids, and total and fecal coliforms are pollutants which the tank was expected to remove and the tank model describes the removal of these parameters. Complete data was desirable on organic and ammonia nitrogen also. Organic nitrogen was expected to be affected but could not be modeled due to a lack of definition between the suspended and soluble fractions. Ammonia nitrogen was not expected to be affected by the tank but complete data was desirable because it represents an oxygen demand in the river.

Table 12. ANALYSES PERFORMED

Parameter	Sewer Monitor Stations	Tank	River Monitor Stations	Locations
Biochemical Oxygen Demand	x	x	x	x
Chemical Oxygen Demand	x	x	x	
pH	x	x	x	
Total Coliforms	x	x	x	x
Fecal Coliforms	x	x	x	x
Nitrogen Series				
Total Organic	x	x	x	x
Ammonia (NH_3)	x	x	x	x
Nitrite (NO_2)	x	x	x	x
Nitrate (NO_3)	x	x	x	x
Phosphate Series				
Total	x	x	x	x
Ortho	x	x	x	x
Solids Series				
Total	x	x	x	
Suspended	x	x	x	
Volatile Suspended	x	x	x	
Cl_2 Demand		x	x	
Chlorides	x	x	x	
Conductivity			x	x
Dissolved Oxygen			x	x
Temperature			x	x

Table 13. SCHEDULE OF ANALYSES
Utilized To Establish Priority of Analyses Based
On Project Requirements

Combined Sewer and Tank Overflow Samples		River Samples
Biochemical Oxygen Demand Suspended Solids Total Coliforms Fecal Coliforms Nitrogen Series Total Organic Ammonia (NH_3)	(A)	Biochemical Oxygen Demand Chlorides Total Coliforms Fecal Coliforms Nitrogen Series Ammonia (NH_3)
Chlorides Phosphate Series Total Ortho Volatile Suspended Solids Cl_2 Demand	(B)	
pH Chemical Oxygen Demand Total Solids Nitrogen Nitrite (NO_2) Nitrate (NO_3)	(C)	pH Chemical Oxygen Demand Nitrogen Series Total Organic Nitrite (NO_2) Nitrate (NO_3) Phosphate Series Total Ortho Solids Series Total Total Suspended Volatile Suspended

The foregoing schedule of analyses was developed after considering past experience in combined sewage characteristics, the importance of each parameter to the tank and river modeling efforts, and the background river data collected prior to 1969.

In developing the river model, parameters utilized included BOD, ammonia nitrogen, total and fecal coliforms, water temperature and dissolved oxygen. Analysis for chloride was specified as a means of quantifying the effect of Lake Michigan sieche activity on the dispersion characteristics of the lower Milwaukee River.

The laboratory staff attempted at all times to conduct a thorough analysis of each sample collected. During extended periods of rainfall or at times of numerous successive storms, an excessive number of samples were generated and delivered to the laboratory. To ease the resulting load on the lab staff, without sacrificing data of importance to the modeling efforts, guidelines were developed for reducing the number of lab tests required.

The first preference was to reduce the individual number of samples to be analyzed by compositing individual samples where this could be done without compromising data. For example, a single composite could be made of all short term individual tank influent flow samples or short term tank overflow samples could be composited to yield one hour composites.

When further reduction in the analytical load was necessary, certain parameters were eliminated from the investigation. Parameters were eliminated in the order of their importance to the study as shown in the schedules in Table 13.

ANALYTICAL PROCEDURES

Experience gained from the analysis of background Milwaukee River samples and "grab" samples of the overflow from the project study area combined sewer system suggested the following analytical methods be employed in the investigation of the parameters of interest to the study.

Biochemical Oxygen Demand	Standard Methods[4] Dilution technique.
Chemical Oxygen Demand	Standard Methods Dichromate reflux method.
pH	Standard Methods Glass-electrode electrometric method.
Total Coliform	Standard Methods Membrane filter procedure using M-Endo broth.

Fecal Coliform	Method as recommended by Geldreich, Clark, Huff and Best[5] MFC medium (Difco) with incubation at 44.5°C.
Total Kjeldahl Nitrogen	Standard Methods
Ammonia Nitrogen	Standard Methods For river samples and for low concentration in combined flow, the direct nesslerization procedure was used. A spectrophotometric analysis was used with a standard curve. Samples with higher concentrations of ammonia were analyzed by the distillation and titration procedure.
Nitrite Nitrogen	Standard Methods Spectrophotometric procedure.
Nitrate Nitrogen	Standard Methods Phenoldisulfonic acid method
Total Phosphate	Standard Methods Nitric acid - sulfuric acid Method.
Ortho Phosphate	Standard Methods Stannous Chloride procedure.
Total Solids	Standard Methods.
Total Suspended Solids	Standard Methods as revised. In lieu of the asbestos mat in a Gooch crucible, a glass fiber filter was used with a 5% Celite solution to seed the filter. Detailed procedure immediately follows this listing.
Total Volatile Suspended Solids	Standard Methods
Chlorine Demand	Standard Methods Laboratory Method.
Chlorides	Standard Methods Argentometric (silver-nitrate) Method.

Conductivity	Specific conductance was measured with a Hellige, Inc. Conductivity Meter. Three ranges were available: 0-50 Micromhos 0-500 Micromhos 0-10,000 Micromhos Procedure consisted of measuring the sample temperature, setting proper temperature on the instrument and reading the micromhos directly with suitable probe immersed in the sample.
Dissolved Oxygen	Standard Methods Azide modification of the iodometric method was used for the laboratory analyses. Samples collected in the field were fixed immediately and titrated following return to the laboratory.
Water Temperature	Field measurement at time of sample collection.

Suspended Solids Determination Procedure

1. Place a glass filter paper (rough side up) at the bottom of the gooch crucible.
2. Seat the glass filter paper with about 1 ml of distilled water.
3. Add 5 ml. of 5% solution of celite (running the solution down the sides of the cricible).
4. Allow celite solution to settle for 2 minutes and then apply vacuum.
5. Place porcelain disc on top of filter and celite mat.
6. Wash the filter and celite with 3 separate 10 ml washings of distilled water.
7. Place in 103°C oven for 24 hours.
8. Place in a 600°C muffle furnace for 20 minutes.
9. Place in a 103°C oven for 24 hours.
10. Place in a desiccator for 1 hour.
11. Weigh the crucible (for tare weight)
12. Filter the sample through the gooch crucible. Quantity of sample to be determined on the basis of how difficult filtration becomes.
13. Place in a 103°C oven for 24 hours.
14. Place in a dessicator for 1 hour.
15. Weigh crucible and sample to obtain weight of the suspended solids in the sample.

The above described suspended solids determination method or similar variations have been adopted at many laboratories throughout the country as an improvement in analysis technique.

DATA HANDLING

Immediately upon analysis, each item of combined sewer and river quality data was recorded in a laboratory notebook by the lab technician. The raw quality data was then transferred to IBM 529 forms. These forms were put through an optical reader which automatically punched the data onto IBM Cards. Duplicate sets of cards were prepared in this manner for distribution. This multiple distribution of data cards provided a guarantee that data would not be lost due to fire, vandalism or the like. Combined sewer flow rate data, after being taken from the monitoring station strip chart recorders, was also punched onto IBM Cards.

The raw data punch cards were then processed to provide a listing of the data distinguishing between river and combined sewer, wet and dry weather quality data and combined sewer flow rate data. The quality data was divided into two parts separating the parameters into two sections to permit printing of the data on normal IBM print out sheets.

Listings of all the data accumulated during this project are referenced in Section XII.

OPERATION AND MAINTENANCE PROBLEMS AND RECOMMENDATIONS

In other sections of this report reference has been made to operating problems encountered in the monitoring and sampling systems employed on this project. Details and an equipment description was given for the monitoring station in Section V and for the Detention Tank components in Section VI.

The following is a discussion of the operating problems. Suggestions and recommendations are presented in order that such problems may be reduced or eliminated on future, similar monitoring systems.

Instrumentation

Operating problems resulted from relatively frequent failure of flow indicating, totalizing and recording equipment, and the related electronic hardware associated with monitoring station control and sampling systems. The degree of difficulty was probably not much greater than that involved in a typical water or wastewater treatment plant incorporating similar equipment. However, such failures in the stations had a much more severe impact on the demonstration project than they might have had on the operation of a treatment plant.

Failure frequently meant the loss, or potential loss, of data which had to be accumulated in a relatively short span of time, and program timing did not, for the most part, include the luxury of retrieving lost data at a subsequent time. (An exception to this condition concerned data lost due to problems with the monitoring stations during the first year of operation. Delay in the construction of the detention tank moved back the project timing which provided extra time to obtain supplemental data from the stations.)

When instrumentation failures occurred, equipment downtime was excessive for the purposes of this study. Delays were encountered in the arrival of manufacturers' service personnel, in obtaining repair parts and in sustaining unit operations even after service visits. Multiple, simultaneous failures caused complex repair problems, not easily remedied by station operating personnel.

For future, similar projects, the following measures should be considered:

1. Provisions should be made to include, well-trained service personnel in the project staff, virtually on a full-time basis if data collection is to be a major function of the project. This can be accomplished by requiring such service as part of the instrumentation supplier's responsibility, or could be part of the Owner's or Engineer's staff. A typical service contract, providing for dispatching personnel from the manufacturer's service organization would not, in most cases, be an adequate arrangement.

2. Arrangements should be made to insure adequate stocking of spare parts. Such parts could be purchased and stored by the Owner, or stockpiled locally by the manufacturer on a guaranteed basis.

3. Consideration should be given to providing backup systems for the most critical system components.

Bell-Type Pressure Switches

Each sewer monitoring station was equipped with a bell-type pressure switch designed to activate the flow measurement system whenever overflow occurred. As the sewer liquid level rises over the mouth of the bell, the column of air trapped in the tubing connecting the bell to the station is compressed. This causes the tripping of a pressure sensitive rubber membrane switch at the station, which, in turn, initiates station operation.

The bells were originally positioned so that the mouth was approximately 1/2 inch below the level of overflow at each station. The resulting 1/2 inch compression of the air column was sufficient to trigger the pressure switch. However, over the four year period of operation the rubber membrane apparantly hardened, losing its sensitivity. This problem was not discovered until the last few months of operation and, therefore, new membranes were not installed as would normally be advised. For the final few months of station operation the bells were lowered 1/2 to 1 inch to provide the compression necessary to trigger the switch.

As the hardening of the rubber membrane could be dependent upon frequency of use, ambient temperature, gases, etc., the useful life of the membrane would vary for different installations. The sensitivity of each switch should, therefore, be checked frequently to insure reliable operation.

Because the bells were located directly in the sewage flow path they were easily clogged by rags, tissue, etc., and required cleaning following each overflow event. Also, under certain flow conditions the bells failed to respond to the rising sewer liquid level. A possible reason for this is that under high flow rates the flow around the bell may have caused a pressure differential between the air in the tubing and the water surging around the bell. With the water pressure below the bell less than the air column pressure, compression of the air column would be impossible.

To eliminate this problem, the pressure bells were relocated to the overflow portion of the chambers. This relocation resolved the problem. Another solution to the problem could be the construction of a stilling well adjacent to the sewer. Availability of a stilling well would also permit the possible use of a float device, or an air bubbler system, in lieu of the pressure bell.

Nitrogen Supply

Nitrogen gas was utilized in the operation of the monitoring station flow measurement bubbler systems and composite samplers

Nitrogen was chosen for use because it would not induce biological action in collected samples. It would also not combine with gases present in sewers to form explosive mixtures.

The use of nitrogen did, however, present some problems in the operation and maintenance of the stations. Replacement of empty cylinders was a difficult and time-consuming task which required the availability of personnel able to lift and carry the heavy cylinders.

A greater problem, directly affecting the data collection effort, was the loss of nitrogen due to leaks in the system. Leaks small enough to evade detection were however large enough to empty a nitrogen cylinder in just a few days. If this was not discovered in time, a small leak in the system could empty a cylinder and eliminate all sampling and flow monitoring until a new tank could be installed. This problem was overcome through a daily visitation schedule established during the final year of testing.

Sampling Systems

Each of the sewer monitoring stations was equipped with an ejector type sampler located either directly in the sewer or in the overflow chamber. It was found that when the sampler rested directly on the floor of the sewer or chamber, sediment clogged the sampler inlet ports or the nylon ball valve. When the ball valve did jam no further sampling could be accomplished. Attachment of 1/2 to 1 inch thick rubber pads to the bottom of the samplers raised them above the sediment and partially alleviated the problem. Clogging still occurred but much less frequently.

The individual vacuum type sampler at the Commerce and Humboldt monitoring station required occasional backflushing of the tubing to remove solids drawn up and trapped in the sample lines. Also, due to the high vacuum necessary to deliver the sample to the monitor station, rags, tissue, etc., were occasionally drawn over the face of the sampler. When this occurred, one or more samples were missed or were drawn through the obstruction with the solids being filtered out. This problem could be largely eliminated by installing automatic backflushing systems, designed to function prior to collection of a sample. However, this problem occurred infrequently, and the effect on the sampling program was negligible.

Level Measurement System

The sewer monitoring stations were equipped with removable, flexible bubbler tube type level sensing devices to monitor sewage flow rates. Plastic bubbler tubes were run through 6 inch diameter access pipes from the monitor station to the sewer where they were connected to metal bubbler tubes anchored to the sewer wall.

At three of the eight sewer monitoring stations locations the plastic tubing was chewed through by rodents and had to be replaced. To eliminate this problem, the flexible plastic tubing could be encased with heavy, flexible, metal conduit for its entire length.

In addition to the foregoing the original instrumentation design included the characterization of the level to flow signal. A motion characterizer, which was a motion balance pneumatic device, was connected to the bubbler line and accepted the same pneumatic signal as the level DP cells. While this characterizer may function well under continuous service, the intermittent operating conditions plus a sometime hostile atmosphere gave many problems including the sticking of moving parts. The use of a function generator operating from the DP level signal may have eliminated many problems and much of the down time encountered.

Another problem involved the anchorage of the bubbler piping system. Metal tubing in sewer was torn from the anchorage during severe rainstorms and sewer flows. Replacement of the piping utilized extra heavy anchors.

Detention Tank Influent Flow Monitoring System

The sewer monitoring station at Commerce and Humboldt was utilized to the greatest extent possible to monitor the tank influent flow rate. The rate of flow was measured through a 60 inch flume by means of a level sensing bubbler tube system. A level proportionate electrical signal was transmitted to the tank where it was converted to flow rate and recorded.

As previously mentioned in this Section, this bubbler system often exhibited erratic behavior during the course of the study. Isolation of various stages of the system for inspection eventually pointed to some fault at the open end of the bubbler tube as being the most probable source of trouble.

By the end of the study the conclusion was drawn that the problem was physical in nature, and related to the position of the bubbler tube in the sewer. Being positioned directly upstream of the flume the tube was subject to the full force exerted by the high rates of flow probably accounting for the resulting pressure drop through the bubbler system.

A feasible answer would be to install a stilling well adjacent to the sewer. The bubbler tube could then be placed in the well, protected from the velocity of flow in the sewer. Because of 1) the expected high cost of installing this system 2) the tenure of this project and 3) the existing backup systems, this theory was not followed to completion.

Because of the importance of maintaining an accurate influent flow record, it is recommended that at least one backup measurement system be provided. To provide influent flow rate data for use in this study, alternate systems including per foot tank volumes converted to flow rates, and a bubbler system to measure and record the head over the overflow weir were used.

Corrosive Gases

Sewer gases were found to vent to the monitoring stations through the cast iron access pipes which carry the sampler, pressure bell and bubbler lines. These gases proved to be corrosive to the copper tubing, valves and fittings housed in the monitoring stations.

Over a 4-year period the copper tubing, aside from discoloring, did not appear to be adversely affected by the bases. However, while the tubing appeared unaffected, gases entering the monitoring stations from the sewers apparently caused the brass fittings to crack thus requiring frequent replacement. Until a cracked fitting could be located and replaced, nitrogen was lost from the system. If the cracked fitting was on a sampler line, the loss of pressure in the line could be sufficient to present sampling and, if on a bubbler line, the loss of pressure could result in the recording of inaccurate sewage flow rate data.

To eliminate this problem, removable, air-tight covers could be required over the station end of the access pipes. The covers must be removable to permit access to the tubing. Project tenure did not warrant further work in this area.

Dissolved Oxygen Analyzers

As previously discussed in this report, the D.O. analyzers installed at the river monitoring stations provided little useful data. The probes originally selected for use proved to be completely unreliable even with concentrated maintenance efforts afforded them. Use of the four probes was eventually discontinued due to the increasing severity of malfunctioning. It appears that no probe available at the time of contract (1969) would have been able to fulfill the requirements. Therefore adjustments were made in data collection to fill the gap in the desired information.

During the project two analyzers of different manufacture were installed at the Cherry Street river monitoring station and their performance was evaluated. Both probes operated satisfactorily and provided accurate data as long as they were well maintained. Unfortunately, maintenance requirements of these probes included daily cleaning and accuracy checks against manually collected samples. This high degree of maintenance was required due to the adverse effect of a scum growth present in the Milwaukee River during the summer months. It is possible that once the river water quality is improved available D. O. probes may work satisfactorily.

For use on this project, the dissolved oxygen analyzers were concluded to be of minimal value because of the expense that would have been involved to maintain the probes or obtain continuous D. O. data. The personnel assigned to maintain the probes could better utilize the time required for maintenance to collect manual samples for titration. Manually collected samples, when handled by an individual who has undergone the proper training, can provide dissolved oxygen data of consistently reliable quality, however the data would not be of the continuous variety.

The good results obtained during the evaluation of the two probes late in the project indicates that in other installations the use of automatic analyzers should be seriously considered, provided water quality permits. Continuous D.O. readings make the task of characterizing the river water quality an easier one but, again, the unique scum growth they were confronted with minimized their worth to this particular study.

Summary

The field of process instrumentation is highly active and progressive. Developments of new components could make some of the foregoing suggestions and recommendations irrelevant in a very short time.

Perhaps the most important concept is that of reliability of systems for monitoring stations such as those involved in these studies as compared to comparable systems in other public works projects. Equipment malfunctions at a wastewater treatment plant, a water treatment plant or some manufacturing facilities may cause inconvenience, or control problems, but the basic plant function is infrequently lost. This is not the case with monitoring stations, intended to function as data collection devices for a short period of time. Instrumentation failures can mean loss of data which cannot be replaced.

In monitoring station programs, suppliers must understand the somewhat unusual nature of reliability requirements. Appropriate planning to insure such reliability must be done and stringent guarantees must be negotiated to assure an absolute minimum of station downtime. Under natural hydrological situations judgment is used to a great extent; therefore an unrealistic desire for accuracy can well be sacrificed for a more dependable flow measurement system. This will enhance judgment decisions even though some error may still be present. The extreme variability of natural phenomena make extremely accurate flow measurement unwarranted.

SECTION VIII

SUMMARY AND ANALYSIS OF DATA

MILWAUKEE RIVER QUALITY

Data on water quality in the Milwaukee River have been developed from the rivers sampling program. Sampling locations are identified in Table 14 and are located schematically in Figure 26.

The following water quality parameters were measured during the program:

Temperature
pH
Conductivity
Chemical Oxygen Demand
Biochemical Oxygen Demand
Dissolved Oxygen
Chlorides
Total Coliform
Fecal Coliform
Kjeldahl Nitrogen
Ammonia Nitrogen
Nitrite Nitrogen
Nitrate Nitrogen
Ortho Phosphate
Total Phosphate
Total Solids
Total Suspended Solids
Total Volatile Solids

A complete listing of the water quality data from the river survey is contained in Appendices II and III. These Appendices have been organized chronologically.

The total inventory of data on river quality developed during this program, has been subjected to a statistical analysis. This analysis has identified the following water quality conditions at each station:

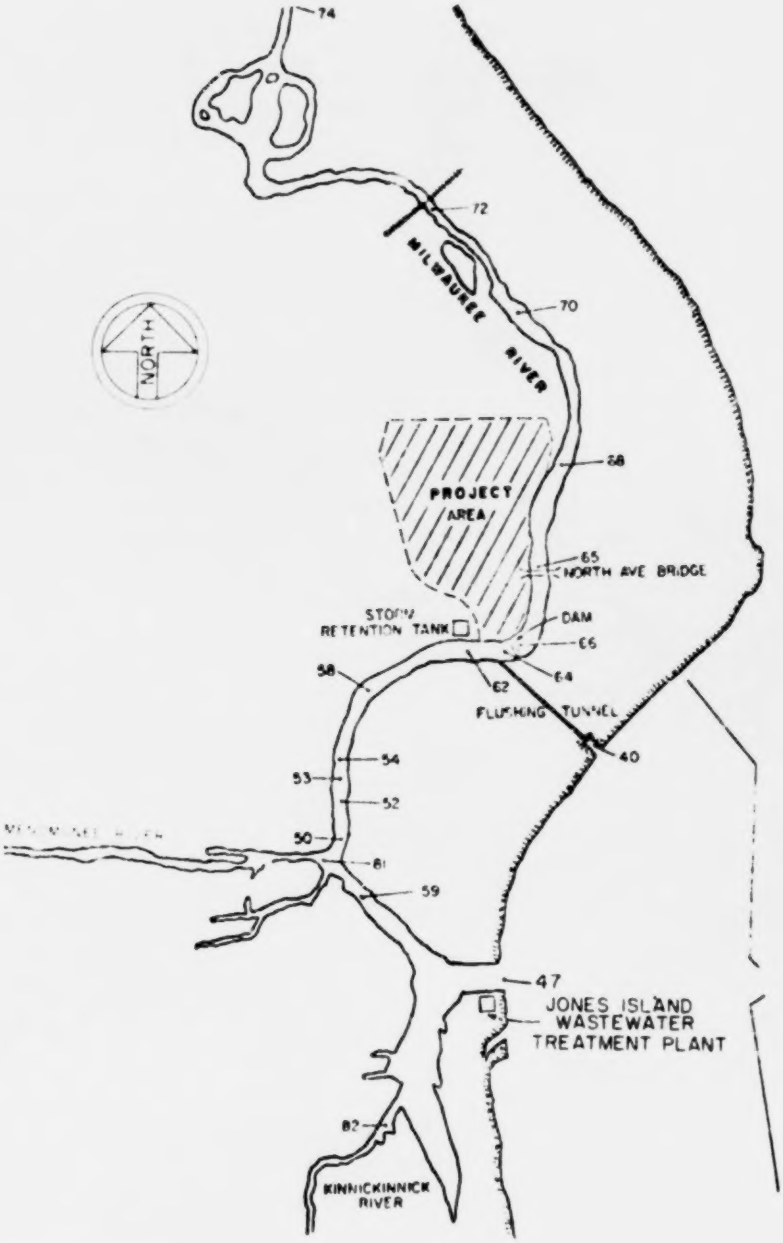
(a) 5 year study period averages
(b) Season averages
(c) Hourly variations for pertinent quality parameters

Table 14. RIVER STATIONS

Lake Michigan (Tunnel Intake)	40
Harbor (off Jones Island STP)	47
Buffalo	50
St. Paul	52
State	53
Kilbourn	54
Cherry	58
South Water	59
Humboldt	62
North Avenue Dam - Upstream	66
North Avenue Dam - Downsteam	64
North Avenue Bridge	65
Locust	68
Capitol	70
Esterbrook Railroad Bridge	72
Silver Spring	74
Menomonee River	81
Kinnickinnic River	82

Fixed Monitor Stations

FIGURE 26- MILWAUKEE RIVER SAMPLING STATIONS



74
72
MILWAUKEE RIVER
70
NORTH
68
PROJECT AREA
65
NORTH AVE BRIDGE
STORM RETENTION TANK
DAM
66
58
62
64
FLUSHING TUNNEL
40
54
53
52
MENOMONEE RIVER
50
81
59
LAKE MICHIGAN
47
JONES ISLAND WASTEWATER TREATMENT PLANT
82
KINNICKINNICK RIVER

In addition, an analysis of the latter diurnal variations in conjunction with rainfall data was performed to identify the effect of rainfall on BOD and suspended solids variations in the river. Table 15 presents a matrix which indicates the locations and periods covered by the river sampling program from which the statistical analysis was made. The identification of seasons as used in this analysis is shown on the table. The particular months assigned to a particular season (e.g., winter equals December, January, February), were made on the basis of generally similar climatic conditions (temperature, rainfall, etc.) rather than the standard calendar designation.

The overall raw water quality (i.e., [illegible]-year average) for each station is illustrated in Tables 16 and 17. These tables provide a general indication of concentration levels of the various constituents in the river. The information should be interpreted cautiously, as the ranges and variation in values at some stations are due to distortions introduced by a limited sampling at a specific time of the year. Temperature variations are a key to such conditions. Coliform data variations are considered to reflect the effect of sample collection during wet versus dry weather periods, rather than specific loads at certain places in the river. The dissolved oxygen variations are related to a number of factors, saturation changes from wide temperature variations in the river, effect of upstream waste sources and storm overflows, and algae production and respiration. From this table and from closer inspection of available raw data, it is not possible to identify significant variations in average quality between upstream and downstream stations. However, this was not the primary purpose of the study, that purpose being to determine the possible improvement in the water quality of the River through the withholding of pollutants generated in the combined sewer area of the City. The effect of the dam impoundment [illegible] that can be seen in the dissolved oxygen surveys reviewed in Section [illegible] since impoundment of algae and pollutants can affect DO levels.

Seasonal raw water quality data are summarized in Tables [illegible] and 1[illegible] for stations 50, 52, [illegible], and [illegible]. These stations were selected on the basis of seasonal distribution and the data availability and were synthesized therefrom. These tables should be interpreted in conjunction with Table 15. Some variation is introduced due to the frequency and distribution of sampling periods. The chloride variation is believed to reflect the impact of road deicing during the winter. Concentrations of chloride are generally two to three times greater during the winter months. Coliform and BOD data are slightly greater during the summer months. The dissolved oxygen variations are due to the same factors mentioned above. Ortho phosphorous and ammonia concentrations varied

Table 15. SEASONAL DISTRIBUTION OF DATA COLLECTION

Station Number	1967	1968	1969	1970	1971	1972
40	A, W	Su, A				S, Su
47						S, Su
50	Su, A, W	S, Su, W				
52		Su, A, W	S, A, W	S, A		S, Su
53		Su, A, W	S			
54	Su, A					
58	Su, A	Su, A, W	S, Su, A	S, A		S, Su
59						S, Su
62		S, Su, A, W	S			S, Su
64			A	A		
65						S, Su
66				S		Su
68	Su, A					
70	Su, A	Su, W				
72	Su, A		S, W			
74		S, W				
81						S, Su
82						S, Su

S = Spring - March, April, May
Su = Summer - June, July, August
A = Autumn - September, October, November
W = Winter - December, January, February

TABLE 16

FIVE YEAR AVERAGE - RIVER WATER QUALITY

Parameter	Station Number							
	40	47	50	52	53	54	58	59
Temperature (°C)	16		9.1	12.1	14.5	17.3	14.7	
pH	8.2	7.8	8.6	8.0	8.1	7.9	8.2	7.9
Conductivity (mmhos)	413	262		575	570		504	452
COD (mg/l)	44		52	42	62	37	53	
BOD (mg/l)	4.8	4.8	6.2	9.6	6.4	8.0	8.6	6.7
Dissolved Oxygen (mg/l)	9.9		7.6	6.3	3.8	6.1	6.9	
Chlorides (mg/l)	20		52	55		25	38	
Total Coliform(nu/ml)	250	6,325	3,050	7,070		33,125	16,000	3,120
Fecal Coliform(nu/ml)	55	35	130	300		473	600	104
Kjeldahl Nitrogen(mg/l)	.97	1.28	1.68	2.54	2.01	1.55	2.14	1.41
Ammonia Nitrogen(mg/l)	.37		1.12	1.29	.86		.84	
Nitrite Nitrogen(mg/l)	.05	.11		.10			.09	.08
Nitrate Nitrogen(mg/l)	.38	.38		.63			.65	.37
Ortho Phosphates(mg/l)	.47	.88	1.01	1.28	1.03	.75	1.15	1.20
Total Phosphates(mg/l)	1.04	1.22		2.63			2.47	1.47
Total Solids (mg/l)				930			436	
Suspended Solids (mg/l)				45			38	
Volatile Solids (mg/l)				27			22	

TABLE 17

FIVE YEAR AVERAGE - RIVER WATER QUALITY

Parameter	Station Number									
	62	64	65	66	69	70	72	74	81	82
Temperature (°C)	12.1				12.8	10.5	11.8	2.3		
pH	8.4	8.5	8.5	8.4	8.1	8.2	8.4	8.6	6.0	7.8
Conductivity (mmhos)	480	985	540	400					412	350
COD (mg/l)	53	180			46	46	34	23		
BOD (mg/l)	5.8	17.6	9.0	9.7	6.1	7.3	5.8	3.1	5.2	5.6
Dissolved Oxygen (mg/l)	9.4				7.7	10.6	10.7	12.9		
Chlorides (mg/l)	34	68	50	47	27	62	27.8	33.6		
Total Coliform (nu/ml)	2,700	3,000	1,050	3,715	35,800	1,015	475	6	742	1,700
Fecal Coliform (nu/ml)	165	17	100	123	450	160	16	2	50	110
Kjeldahl Nitrogen (mg/l)	1.57	6.25	2.11	2.02	1.77	1.96	1.24	1.36	1.24	1.14
Ammonia Nitrogen (mg/l)	.71	1.70		.70	1.0	1.08		.90		.60
Nitrite Nitrogen (mg/l)	.09		.10	.10					.09	.07
Nitrate Nitrogen (mg/l)	.62		.95	.61					.51	.33
Ortho Phosphates (mg/l)	1.01	5.2	1.60	1.34	1.22	1.25	1.15	1.73	1.07	.83
Total Phosphates (mg/l)	1.47	6.	2.03	1.93					1.33	1.06
Total Solids (mg/l)	380		438	450						
Suspended Solids (mg/l)	23	173	21	25						
Volatile Solids (mg/l)	14	34	12	13						

TABLE 18

SEASONAL RIVER WATER QUALITY

Parameter	Station 50				Station 52			
	Spring	Summer	Autumn	Winter	Spring	Summer	Autumn	Winter
Temperature (°C)	7.4	19.8	7.4	2.9	9.7	20.5	15.4	2.4
pH	8.4	8.0	8.4	8.2	8.1	7.8	7.9	8.1
Conductivity					540			
COD (mg/l)	43	38	44	50	42	72	28	37
BOD (mg/l)	5.1	7.5	5.1	5.8	6.5	7.9	5.3	7.8
Dissolved Oxygen(mg/l)	7.7	4.3	7.7	9.3	8.7	2.7	2.7	10.2
Chlorides (mg/l)	29	29	29	79	36.	29	44	112
Total Coliform(nu/ml)	220	7,600	220	175	3,150	12,050	7,350	1,539
Fecal Coliform(nu/ml)	15	360	14	50	160	500	340	257
Kjehdahl Nitrogen(mg/l)	1.01	2.12	1.01	1.82	2.19	2.0	2.05	2.65
Ammonia Nitrogen(mg/l)		.72		1.61	1.08	.68	1.09	1.61
Ortho Phosphates	.98	.79	.98	1.23	.729	.87	1.17	1.54

TABLE 19

SEASONAL RIVER WATER QUALITY

Parameter	Station 58				Station 62			
	Spring	Summer	Autumn	Winter	Spring	Summer	Autumn	Winter
Temperature (°C)	9.8	22	14.5	1.7	9.8	20.	12.4	2.4
pH	8.3	8.	8.2	8.5	8.4	8.2	8.3	8.4
Conductivity					560	410		
COD (mg/l)	45	52	36	28	43	65	59	48
BOD (mg/l)	5.1	6.6	5.9	3.8	6.0	6.6	5.1	4.5
Dissolved Oxygen(mg/l)	9.5	5.1	5.4	12.1	10.2	7.9	8.3	12.6
Chlorides (mg/l)	32	29	43	56	31	27	35	49
Total Coliform(nu/ml)	5,650		400		1,200	6,200	2,250	1,370
Fecal Coliform(nu/ml)	245		23		130	190	146	230
Kjeldahl Nitrogen(mg/l)	1.94	2.15	1.90	1.58	1.87	1.39	1.38	1.78
Ammonia Nitrogen(mg/l)	.91	.64	.87	.83	.60	.55	.66	1.30
Nitrite Nitrogen(mg/l)					.06	.11		
Nitrate Nitrogen(mg/l)					.53	.69		
Ortho Phosphates(mg/l)	.66	.89	1.16	1.36	.86	1.00	1.02	1.38
Total Phosphates(mg/l)					2.45	1.40	1.02	1.21
Total Solids (mg/l)					450	312		
Suspended Solids (mg/l)					22	25		
Volatile Solids (mg/l)					16	11		

from 0.5 to 1.5 mg/l, with the higher levels occuring during the colder months. This variation could be the result of algal requirements during the warmer periods of the year, where active growth tends to strip these nutrients from solution. Concentrations of these nutrients are quite high, and in a range well able to support significant levels of algal growth. Both D.O. diurnal variation and visual observations confirm the presence of significant levels of algae.

At certain times during the program, samples were collected at frequent intervals to determine hourly variation in water quality. Table 20 lists the stations, seasons, and years when such information was collected. Actual average hourly data is listed in Appendix III, where the raw data has been arranged by season and station. No normal variation in quality over the course of a day is evident in the data, nor are any trends indicated. With the exception of coliform concentrations, which exhibit considerable variation, concentrations of contaminants remained relatively constant over the course of a day.

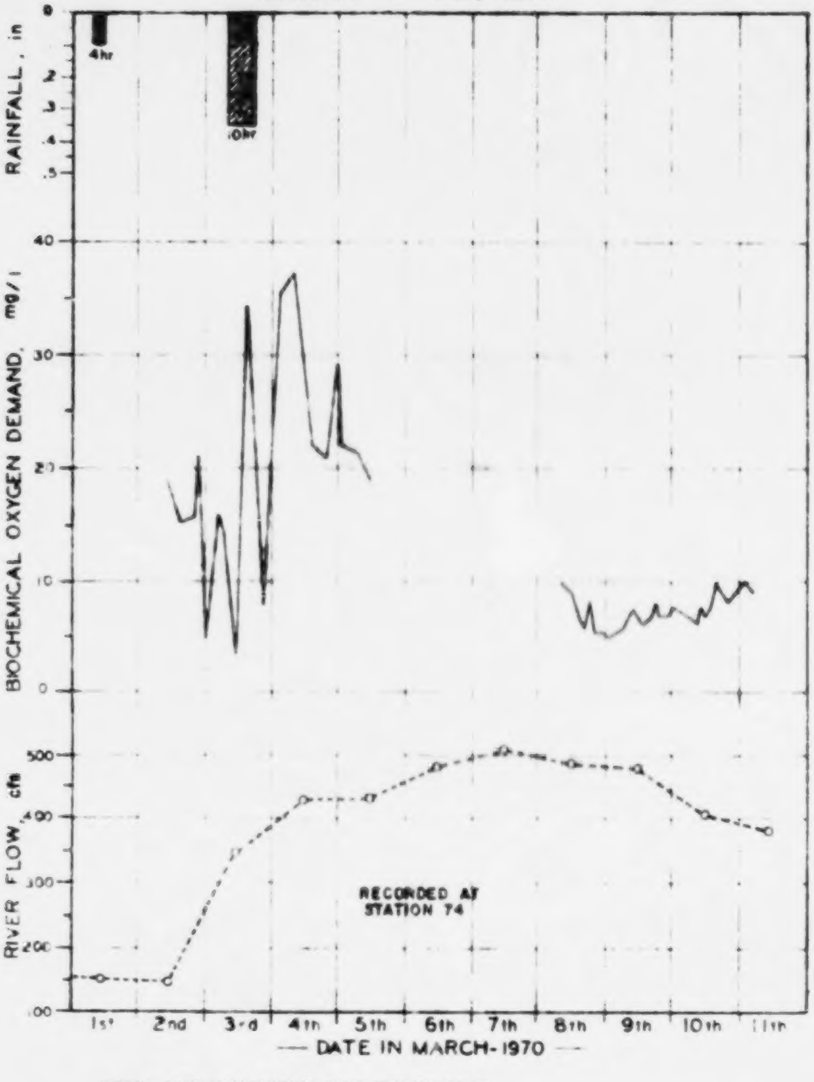
The effect of rainfall and urban storm water overflow on general background river water quality has been explored to a limited degree by studying data developed from the general river sampling program. The long term quality data, obtained from daily samples has not yielded any definitive information. The primary reason appears to be that the water quality in the Milwaukee River is controlled to a significant extent by conditions within the upper reaches of its drainage area and certainly in those areas substantially outside the test area. The occurance of storm events in the drainage basin either unrelated to or poorly correlated with similar events in the test area, the time of concentration for such runoff to reach the test area, and the variability of river flows, makes any analysis of river quality response quite complex and strongly influenced by conditions which were beyond the scope of this project.

One analysis was made using data obtained from daily river quality sampling program, where the sampling period coincided with a storm event. Table 21 lists those events where such data exists. Figures 27 and 28 plot BOD and suspended solids variations for the period March 3 to 11, 1970. (Data for March 6th and 7th was not available due to equipment malfunction or sample handling problems.) This illustrated graphically the water quality response which resulted from this storm event. The storm event in question was a significant one, resulting in a total accumulation of 0.35 inches over a 10 hour period. Quality measurements of BOD and suspended solids at Station 52 (St. Paul Avenue), in the lower reaches of the river show concentrations well above normally observed levels, and increase further at the time of the

Table 20. PERIODS WHEN SAMPLES WERE COLLECTED AT FREQUENT INTERVALS TO STUDY HOURLY VARIATION IN RIVER QUALITY

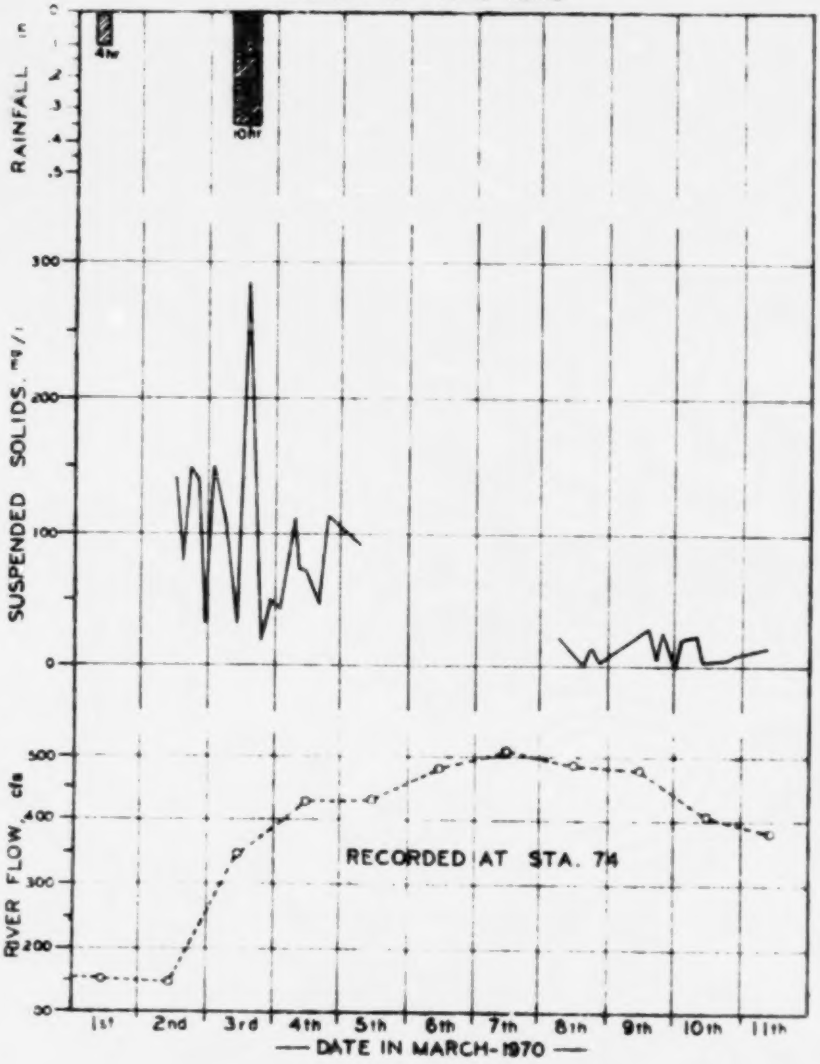
Season	Station Number	Year
Spring		
A	40	1970
B	52	1970, 1972
C	58	1970, 1972
D	59	1972
E	62	1972
F	65	1972
G	66	1970
Summer		
A	40	1972
B	52	1972
C	58	1969, 1972
D	59	1972
E	62	1972
Autumn		
A	46	1967
B	50	1967
C	52	1969, 1970
D	54	1967
E	58	1967, 1969, 1970
F	59	1970
G	62	1967, 1970
H	64	1969

FIGURE 27 - RIVER QUALITY AT STA. 52 VS RAINFALL
MARCH 1st - 11th, 1970



NOTE: DATA FOR MARCH 6 & 7 UNAVAILABLE

FIGURE 28- RIVER QUALITY AT STA 52 VS RAINFALL
MARCH 1st - 11th., 1970



NOTE: DATA FOR MARCH 6 & 7 UNAVAILABLE

storm event on March 3rd. The precipitation shown on March 1st is indicated by weather bureau records to be snow (1.1 inches), and followed a month where no significant precipitation was recorded. The sharp increase in river flow between the 2nd and 3rd may in fact represent either snow melt (average daily temperature between February 21st and 24th was above freezing) or precipitation in the upper drainage basin of the Milwaukee River.

Table 21. RIVER SAMPLING AND STORM EVENT ANALYSIS DATED

Station	Dates
52	March 3 - 11, 1970
58	May 20 - 27, 1970
52	October 12 - 14, 1969
52	October 27 - 28, 1970

The March 1970 conditions may be compared with several others for which comparable data is available. Comparable data is presented in Figures 29 and 30.

From an inspection of such data, it does appear that storm overflows in the immediate area can have a significant impact on river quality. However, significant fluctuations also are observed during periods when no overflows occur, and in some cases (the storm of Sunday, May 24th, 1970 for example), no apparant effect can be discerned. This behavior could be affected by the time of day and day of the week. River quality in the vicinity of the combined sewer overflows in the lower reaches of the Milwaukee River is influenced as much or more by events occuring well upstream of the area of study. Because of all of these complexities, any effort to evaluate river quality responses to storm overflows from the combined sewer system, using the river quality data discussed in this section without the aid of a mathematical computer model, would be highly speculative. Therefore, in order to obtain a reliable quantitative assessment of the effect of urban storm runoff on water quality in the lower reach of the Milwaukee River, the mathematical model developed during this program must be relied upon to characterize water quality responses. The results of this modeling effort are discussed in a later section of the report.

The Milwaukee River, in the area being considered, is subject to wide variations in flow which can occur over relatively short intervals. Table 22 presents a condensed summary of flow data for the lower Milwaukee River. Some 50 years of record indicate a discharge of 45 cfs to be exceeded 95 percent of the time; 172 cfs at 50 percent of the

FIGURE 29 - MILWAUKEE RIVER QUALITY CHANGES DUE TO RAINFALL

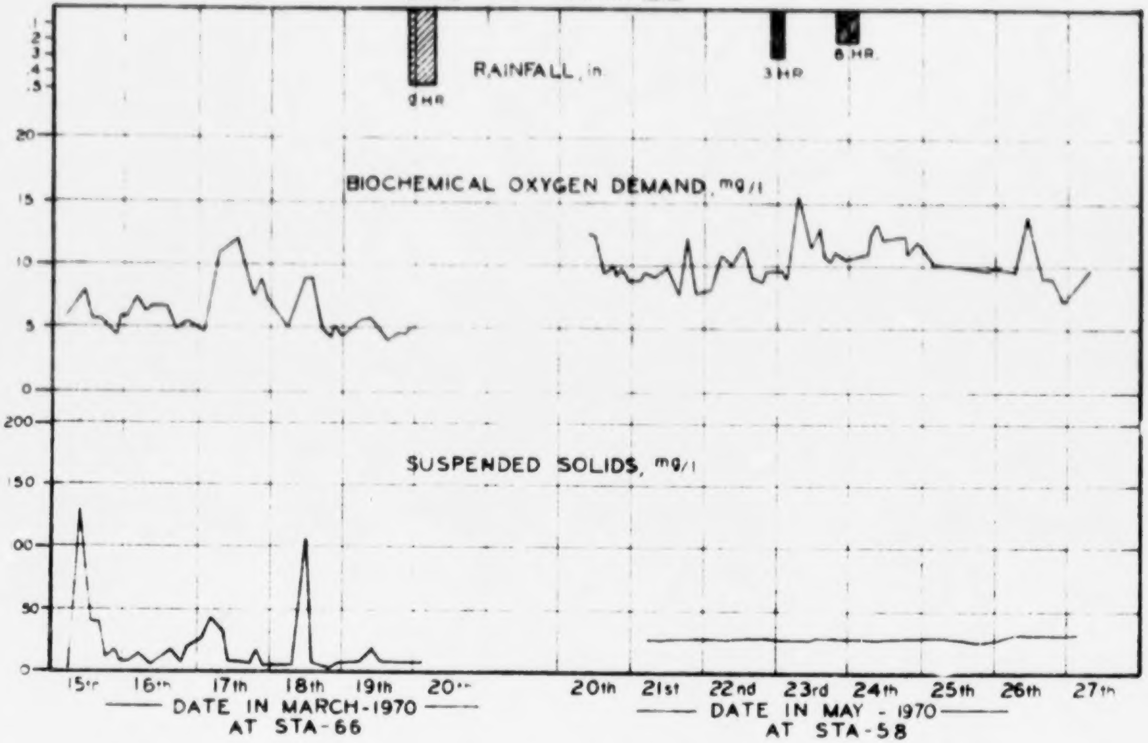
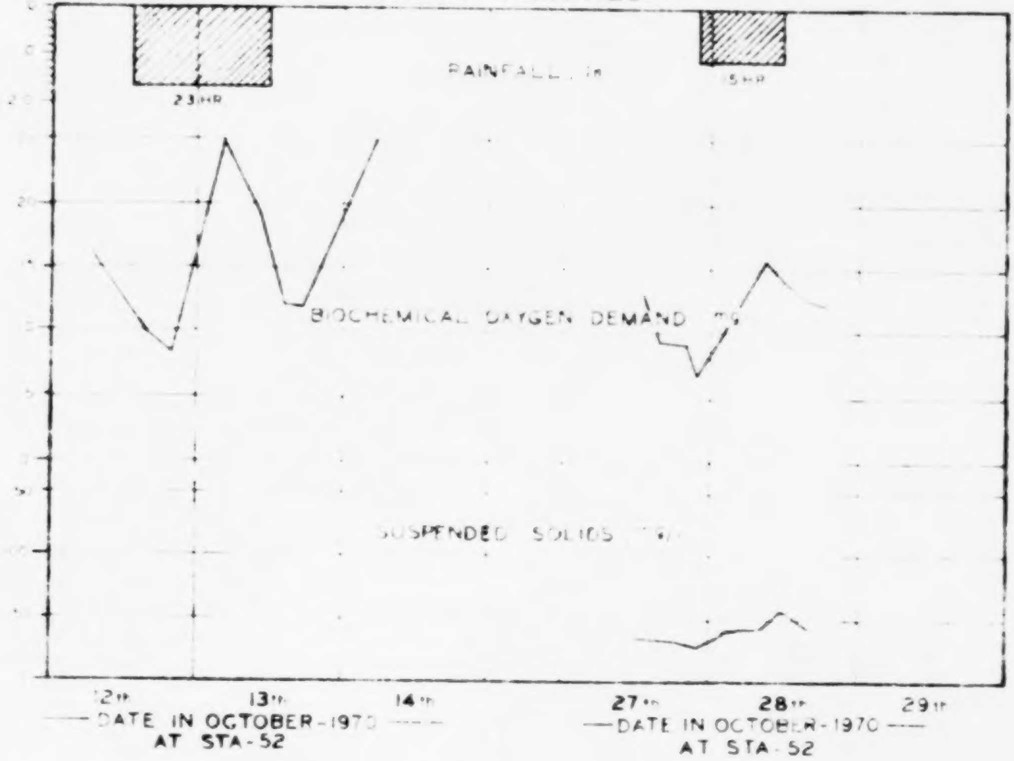


FIGURE 30 - MILWAUKEE RIVER QUALITY CHANGES DUE TO RAINFALL



time; and 1,400 cfs 5 percent of the time. Bi-hourly flows in excess of 3,000 cfs are indicated.

In general, river quality data indicates that the Milwaukee River in the study area contains relatively high levels of polluting material, which are evidentally attributable to both storm water discharges from sewer overflows, and upstream runoff which enters the lower Milwaukee River estuary. Typically, the Milwaukee River exhibits the following characteristic concentrations:

Temperature = 0 - 22°C
pH = 7.5 - 8.5
COD = 35 - 60 mg/l
BOD = 5 - 10 mg/l
Chlorides = 20 - 50 mg/l
Total Nitrogen = 1 - 2 mg/l
Total Phosphorus = 1 - 2 mg/l
Suspended Solids = 20 - 50 mg/l
Total Coliform = 200 - 40,000 nu/ml

Table 22. FLOW RECORD - MILWAUKEE RIVER

Year	Average	Minimum	Maximum
1967	337	52	2,300
1968	255	58	1,620
1969	386	73	3,150
1970	243	43	1,900

Average discharge - 56 years, 379 cfs.

Data reported in cubic feet per second (cfs).

SEWAGE - DRY WEATHER QUALITY

Data taken at intervals during the program have been analyzed to identify quality characteristics of the sewage in the project area during periods of dry weather. A tabulation of all data collected is presented in Appendix IV.

Table 23 identifies those periods during the program in which suitable dry weather data was obtained. On many occasions during the program, sampling efforts intended to produce dry weather quality were not usable for that purpose because of storm events which occurred during scheduled "dry weather" surveys.

Table 23. DATES OF DRY WEATHER SEWAGE SAMPLES

Sampling Stations	Dates of Dry Weather Samples		
	1969	1970	1972
49	June 2-13	January 14 to February 3	
	July 13-14		
	July 30 to August 11		
	December 17-18		
21		February 11-23	March 24 to August 9

These data represent daily average from 24-hour composite. All other data based on 1-to [illegible]-hour composites.

The entire body of dry weather quality data was analyzed, and a summary of average, maximum, and minimum concentrations observed in the sampling program is presented in Table 24. Average concentrations appear somewhat low for "typical" domestic sewage, and rather wide variation between maximum and minimum values are indicated. Some of the extreme values may be due to the effect of wet weather conditions, such as residual runoff or effects of limited infiltration. This explanation, however, is speculative, and no reliable explanation of the range of concentrations observed is available. Although some of the variation is also due to anticipated seasonal and daily variation (including weekday and weekend), these do not account for the observed variations. Some of the higher values, especially in chlorides, are the result of snow melt runoff during winter thaws in otherwise dry weather conditions.

Because of the large number of individual observations for most parameters, the average values reported are considered to be representative of "typical" dry weather quality in this system.

Seasonal variations in dry weather quality were examined by sorting data by season, and analyzing data to determine mean concentrations and standard deviation for each parameter. This analysis is summarized in Table 25.

Spring seasonal averages are somewhat limited by the relatively small number of observations, and must be evaluated with this in mind. Both winter and summer data have a sufficient number of individual data observations to provide a reliable comparison. A seasonal average for the fall months is not presented because of data limitations. Efforts to supplement the deficiency of data from prior sampling programs, during 1972 were frustrated by the extreme wet weather conditions during this period.

Contaminant concentrations during the winter are higher than summer values for all parameters, except phosphate and total solids. The magnitude of the standard deviations from mean values attests to the rather wide variability between individual observations. Spring seasonal data shows the highest concentration of all parameters, however, the number of observations represented compared with those for winter and summer averages suggests caution in drawing firm conclusions.

Diurnal quality fluctuations were explored using winter and summer seasonal data breakdowns. Data was sorted on the basis of hour of the day during which the observation was made. Table 26 lists average values for each quality parameter, each hour of the day for all winter data. Table 27 does the same for summer observations. Much of the quality data does not show any clear-cut diurnal pattern, beyond generally

Table 24. DRY WEATHER SEWAGE QUALITY

Parameter	Number of Observations	Average Concentration	Maximum Concentration	Minimum Concentration
pH	433	7.6	8.7	6.7
COD (mg/l)	382	238	730	35
BOD 5-day (mg/l)	427	112	323	17
Chloride (mg/l) Cl	98	141	826	3
Total Coliform (nu/ml)	338	930,000	-	11,000
Fecal Coliform (nu/ml)	316	38,000	540,000	400
Total Kjeldahl Nitrogen (mg/l-N)	161	20.7	47	5.6
Ammonia Nitrogen (mg/l-N)	160	11.7	31.2	1.0
Nitrate (NO_3)(mg/l-N)	19	0.5	1.6	0.1
Nitrite (NO_2)(mg/l-N)	2	0.01	0.01	0.01
Orthophosphate (mg/l-PO_4)	159	10.7	39	1.0
Total Phosphate (mg/l-PO_4)	158	18.4	60	0.5
Suspended Solids (mg/l)	149	150	900	5
Volatile Suspended Solids (mg/l)	148	115	700	2

TABLE 25

DRY WEATHER CONDITIONS – AVERAGE SEWAGE QUALITY

	Spring			Summer			Winter		
Parameter	Average	Std.Dev.	# Events	Average	Std.Dev.	# Events	Average	Std.Dev.	# Events
pH	7.4	.37	12	7.7	.37	256	7.6	.28	165
BOD (mg/l)	169	58	13	94	41	251	133	67	163
COD (mg/l)	309	71	13	206	104	252	301	154	117
Chlorides (mg/l)	162	29	13	51	33.7	7	146	147*	78
Coliforms (nu/ml)									
Total	243,000	87,475	13	261,884	285,152*	159	1,629,270	1,628,890*	166
Fecal	21,546	8,248	13	19,887	39,074	137	54,899	69,494*	166
Nitrogen (mg/l-N)									
Kjeldahl	26.7	2.8	13	18.7	6.2	64	21.4	9.2	84
Ammonia	17.6	2.4	13	9.8	3.0	64	12.2	6.1	83
Nitrite	-	-	-	.01	.000003	2	-	-	-
Nitrate	.48	.41	13	.39	.25	6	-	-	-
Phosphate (mg/l-PO$_4$)									
Total	33.4	10.7	13	19.4	12.2	81	15.3	8.1	81
Ortho	22.0	4.6	13	11.5	7.2	82	8.4	4.7	82
Solids (mg/l)									
Total	845	97.5	13	471	76.7	5	128	0	1
Suspended	155	55.4	13	122*	135*	51	145	41.2	85
Volatile Suspended	110	33.6	13	87	103*	51	123	70.8	84
	March, April, May			June, July, August			December, January, February		

*Standard Deviation greater than average.

TABLE 26

DRY WEATHER SEWAGE QUALITY
DIURNAL VARIATION - WINTER SAMPLES

Hour	pH	COD mg/l	BOD mg/l	Cl mg/l	TCOL nu/ml x 1000	FeCl mg/l	Kjeld. mg/l	NH$_3$-N mg/l	Ortho. mg/l	TPO$_4$- mg/l	TSS mg/l	TVS mg/l
1	7.4	193	112.2		1,005	22						
2	7.4	182	156.3		574	45						
3	7.5	178	122.8	83	651	16	16.83	11.00	6.40	13.08		
4	7.6	167	78.2		970	67						
5	7.7	116	70.5	90	105	3	14.55	8.13	3.90	8.55	91	78
6	7.6	174	86.8	105	835	33	17.21	9.46	6.34	12.41	92	79
7	7.7	57	44.9		50	10						
8	7.5	239	63.9	80	255	3	10.64	5.50	1.35	4.00	77	67
9	7.7	245	89.6	147	324	9	12.68	7.33	3.62	6.90	92	77
10	7.6	304	110.0	747	1,693	75	15.68	11.40	8.00	17.00	113	88
11	7.5	265	144.1	79	2,041	52	28.84	17.75	12.33	21.32	131	116
12	7.5	327	143.7	95	1,916	51	24.35	15.95	7.65	14.30	153	131
13	7.7	322	157.1	89	1,536	51	29.79	17.25	14.45	25.60	131	126
14	7.5	393	163.8	170	2,004	97	23.86	12.78	6.86	12/50	168	147
15	7.6	344	146.6	132	1,939	76	21.41	13.06	8.67	16.64	113	86
16	7.6	376	174.3	72	2,481	58	30.24	13.60	17.80	35.60	123	114
17	7.5	371	149.2	209	2,192	53	18.47	9.61	7.87	15.90	224	181
18	7.4	374	151.1	259	2,242	74	20.66	10.71	6.98	10.70	211	166
19	7.6	323	125.1	54	1,663	93	33.46	17.30	15.95	31.00	165	155
20	7.5	427	135.3	180	2,204	75	21.62	12.74	7.24	14.51	175	145
21	7.5	283	130.0	221	2,136	70	19.73	10.79	9.32	17.07	140	117
22	7.5	320	158.7	119	3,081	76	26.84	15.89	13.50	24.55	175	160
23	7.5	290	142.0	89	1,914	50	19.82	10.89	9.22	17.92	138	121
24	7.7	277	143.1	92	1,106	21	25.84	14.57	9.72	18.31	159	137

TABLE 27

DRY WEATHER SEWAGE QUALITY
DIRUNAL VARIATION - SUMMER SAMPLES

Hour	pH	COD mg/l	BOD mg/l	TCOL nu/mlx1000	FeCl mg/l	Kjeld. mg/l-N	NH$_3$-N mg/l-N	Ortho. mg/l-PO$_4$	TPO$_4$ mg/l	TSS mg/l	TVS mg/l
1	7.6	247	94.0	108	8						
2	7.7	158	61.0	105	15					8	2
3	7.6	176	136.6	283	9	21.94	9.87	6.97	13.70	100	80
4	7.6	169	84.2	99	22						
5	7.6	182	56.9	149	3	12.81	7.07	3.73	4.60	202	132
6	7.8	163	51.9	310	15	14.17	8.69	6.34	9.60	55	37
7	7.7	100	48.3	152	7	12.32	7.49	5.33	8.60	38	24
8	7.7	85	50.2	190	12						
9	7.7	170	65.3	240	22					56	22
10	7.8	232	104.1	373	21	25.01	13.44	14.44	24.06	176	118
11	7.7	288	116.0	378	25	21.18	11.21	7.62	13.02	160	133
12	7.7	224	110.1	202	11	22.62	12.63	14.55	24.27	100	42
13	7.7	191	95.3	175	17	18.47	10.93	12.70	21.13		
14	7.6	217	93.1	334	25	18.89	9.56	23.20	33.73	92	75
15	7.6	211	104.7	255	20	16.89	8.63	14.83	28.53	93	65
16	7.7	251	105.0	271	20	16.80	8.60	7.30	8.00	162	102
17	7.7	253	93.7	236	14	15.50	7.97	10.67	19.30	39	33
18	7.5	229	104.6	166	11	17.56	9.53	14.39	26.86	56	40
19	7.6	232	133.7	287	19	24.74	9.85	16.52	29.40	367	280
20	7.6	270	123.0	384	94	17.60	10.00	23.00	40.60	164	110
21	7.6	249	120.-	186	12					62	34
22	7.6	211	99.9	268	19	17.70	9.55	16.50	24.00	84	55
23	7.7	233	115.2	402	19	20.40	10.20	5.50	29.20	125	51
24	7.6	203	94.6								

lower concentrations during early morning hours. The plots, Figures 31 and 32 show those parameters which show the most evident diurnal variations. Total phosphate shows a distinct pattern in both winter and summer data, with early evening values approximately three times early morning concentrations. Total coliforms show a clear diurnal variation in winter, but not in summer. This is also the case with nitrogen and suspended solids data.

Chloride concentrations in winter dry weather flow suggest an increase from approximately 100 mg/l during early morning hours until midday - to about twice this concentration during afternoon and evening hours. Since the concentrations begin to increase much later in the day than is observed with the other constitutents, the effect is more probably due to snow and ice melt than to increased sanitary discharges to the sewers. Based on a dry weather flow of about 2 mgd and a chloride increase of 100 mg/l over background concentrations occurring between 2 PM and 10 PM, a daily flushing of about 600 pounds chlorides per day per square mile during dry weather is indicated, which is in excess of the amount of chlorides present in the sewage during non-winter months. The data shows a value at 10 AM which reaches 747 mg/l Cl. This appears to be an abnormal condition occasioned by a measurement during a heavy thaw, and is considered to be not typical of dry weather conditions. However, a 1 to 2 hour runoff under these conditions would contribute approximately 500 pounds Cl per square mile.

A further analysis of dry weather quality data was made to determine whether variations in concentration were present as a function of time elapsed since the antecedent storm event. This data is presented in Figures 33 and 34. If we assume that the low concentrations at time zero in several cases are due to the presence of storm runoff at the time the "dry weather" sample was taken, then the data show no significant effect on dry weather quality as a result of an antecedent storm. This data also suggests that no significant infiltration occurs in the test area. Where an appreciable degree of infiltration is present, one would expect to see a gradual increase in concentrations as the interval to the antecedent storm increases. Kjeldahl nitrogen concentrations show such a pattern, but other quality parameters do not support the actual presence of such an effect. While some of the data imply a decline in concentration with time, the observed differences are well within the normal variations observed for dry weather quality.

FIGURE 31- HOURLY VARIATION – DRY WEATHER SEWAGE QUALITY
WINTER DATA

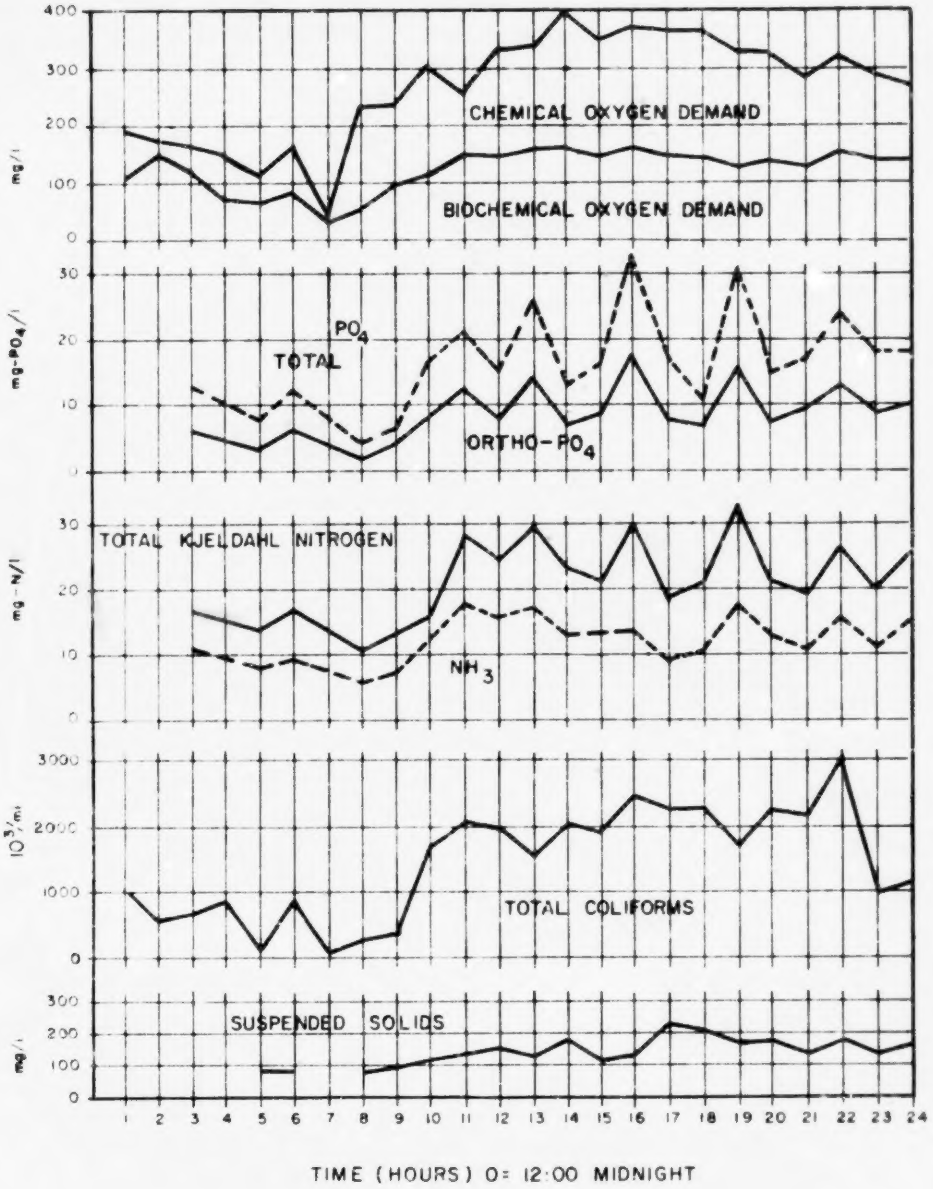


FIGURE 32 — HOURLY VARIATION — DRY WEATHER SEWAGE QUALITY
SUMMER DATA

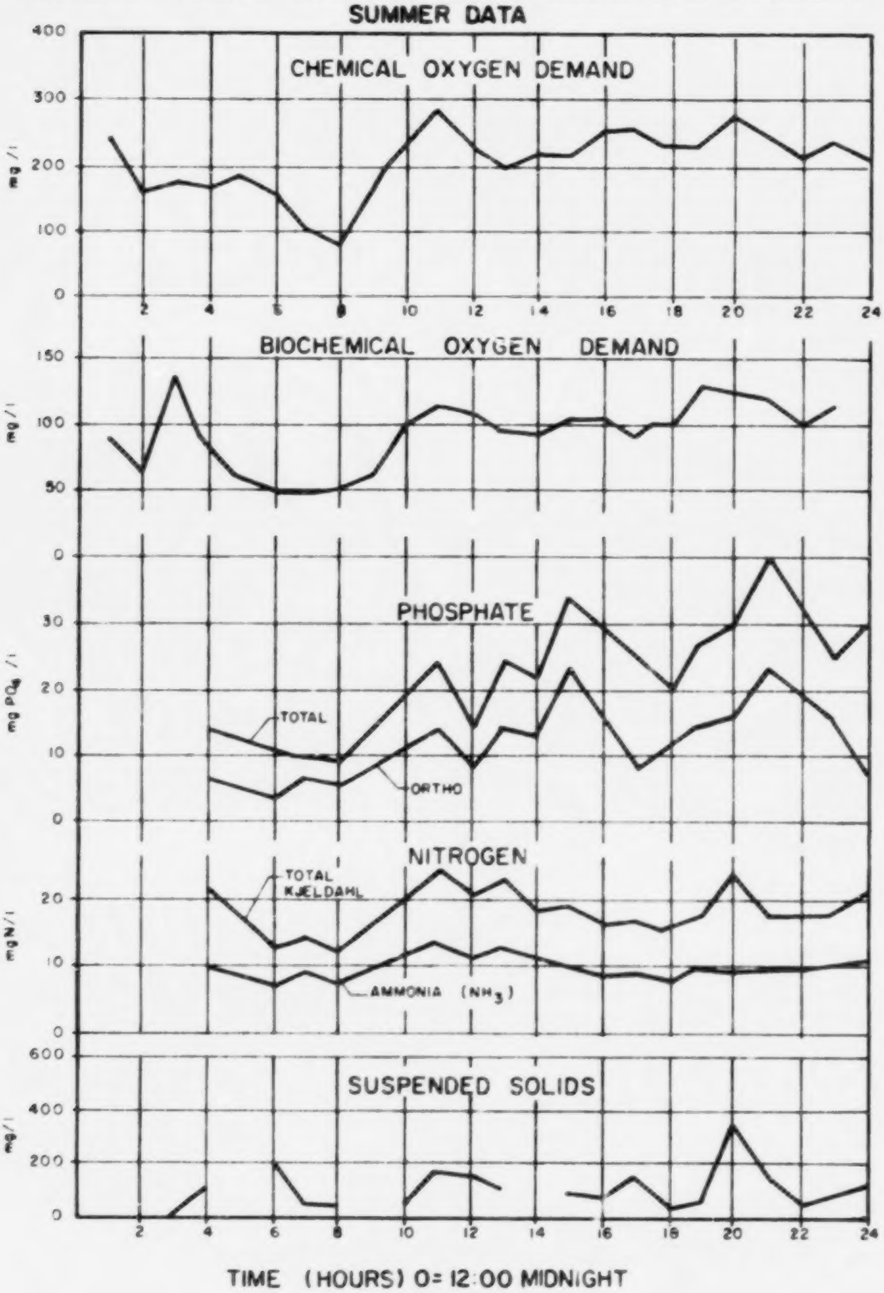
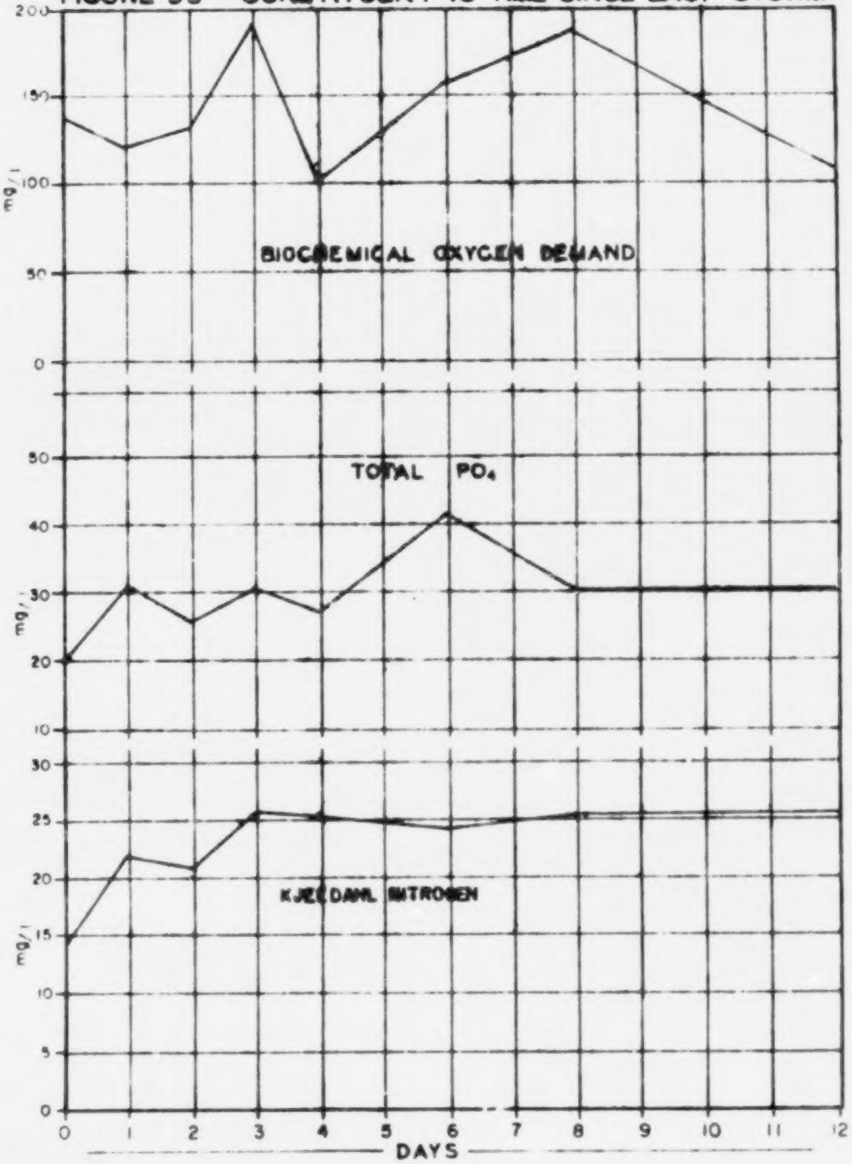
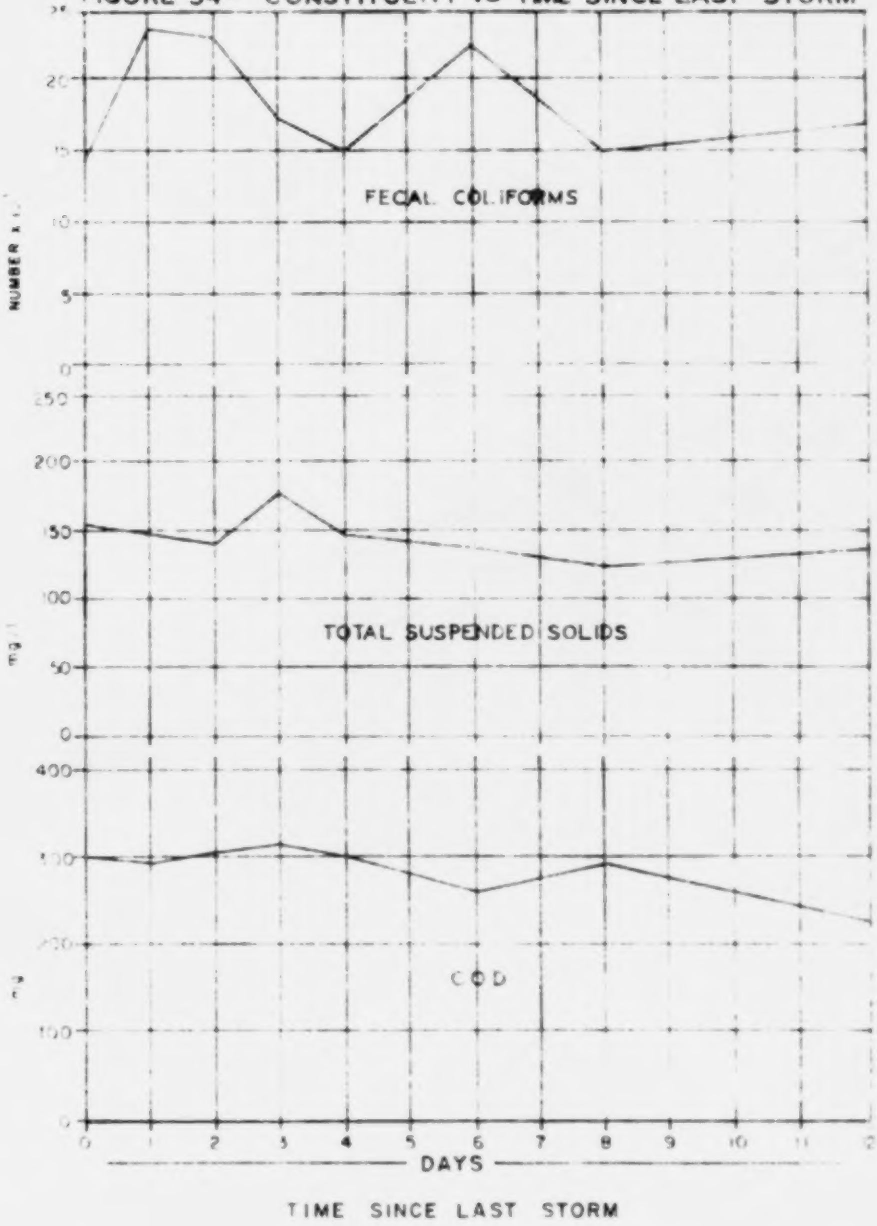


FIGURE 33- CONSTITUENT VS TIME SINCE LAST STORM



TIME SINCE LAST STORM

FIGURE 34- CONSTITUENT VS TIME SINCE LAST STORM



SEWAGE - DRY WEATHER FLOW

Flow rates were monitored at sewer station 21 at various times throughout the program in periods of dry weather, in order to establish normal dry weather flow conditions.

The flow data developed indicates average dry weather flows in the test area range between approximately 1.7 and 2.3 mgd. Sewage flows appear to remain relatively uniform over a period extending from 7 or 8 AM until 8 to 10 PM. Flows then decrease to a minimum which is reached at 1 or 2 AM, and remain relatively uniform until approximately 5 AM. Maximum flows are approximately 100% greater than minimum flows during a 24 hour period. However, wide variation between maximum and minimum within hourly flows has not been observed.

Figure 35 plots schematic representations of daily dry weather flow variations based on data collected during fall months (September, October, November) and spring months (March, April, May). The fall average shown is slightly greater than that observed in spring, although the difference is small; approximately 0.5 mgd. Winter dry weather data is not presented here, as it proved to be influenced in an erratic manner by snow melt under otherwise dry weather conditions. An analysis of data indicates that snow melt conditions increased "dry weather" flows by between 0.3 and a maximum of 1.5 mgd and is dependent upon the amount of snow cover, temperature and ice control conditions.

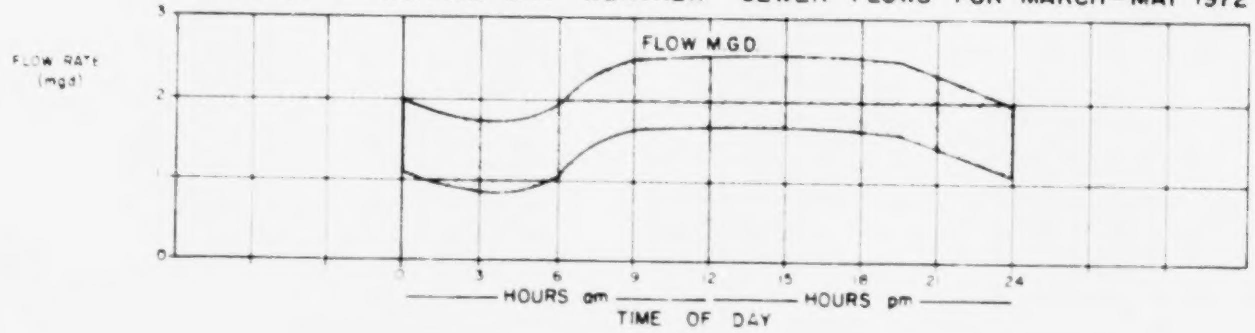
One set of dry weather data, that for the sampling date September 18, 1970, is plotted in Figure 36. This illustrates in detail the data presented in schematic form in the previous figure. In addition, this particular set of data was commenced immediately after a significant storm event, and illustrates a relatively rapid return to essentially dry weather flow conditions.

Variations in dry weather flow, either due to daily or seasonal fluctuations are not considered to be a significant factor in the analysis and modeling of the storm overflows in the test area. Observed variations are relatively small, particularly so when compared with the combined flows required to cause an overflow. Although winter flows are somewhat higher and more variable, the significant storms, i.e., those generating significant overflows are relatively rare occurrances in winter.

RAINFALL

The U. S. Weather Bureau's narrative climatological summary for Milwaukee provides the following comment:

FIGURE 35 - AVERAGE DRY WEATHER SEWER FLOWS FOR MARCH-MAY 1972



AVERAGE DRY WEATHER SEWER FLOWS FOR SEPT.-NOV.

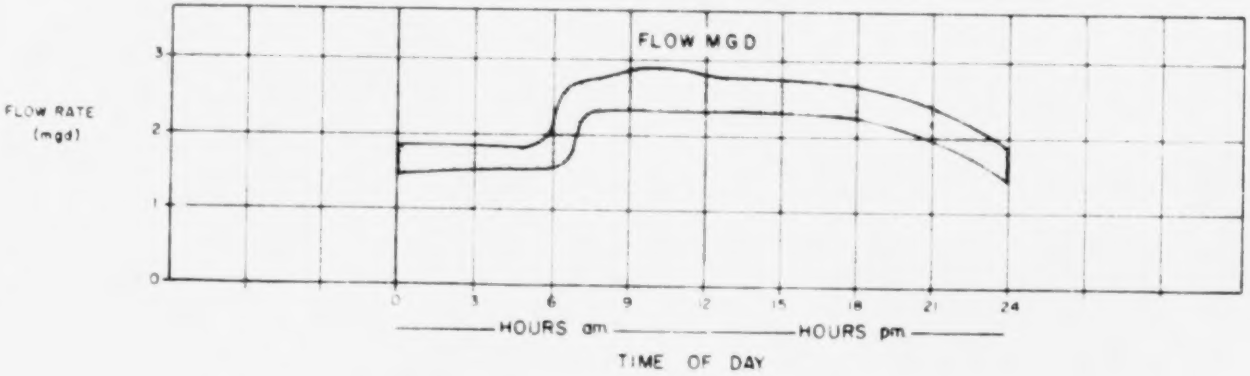
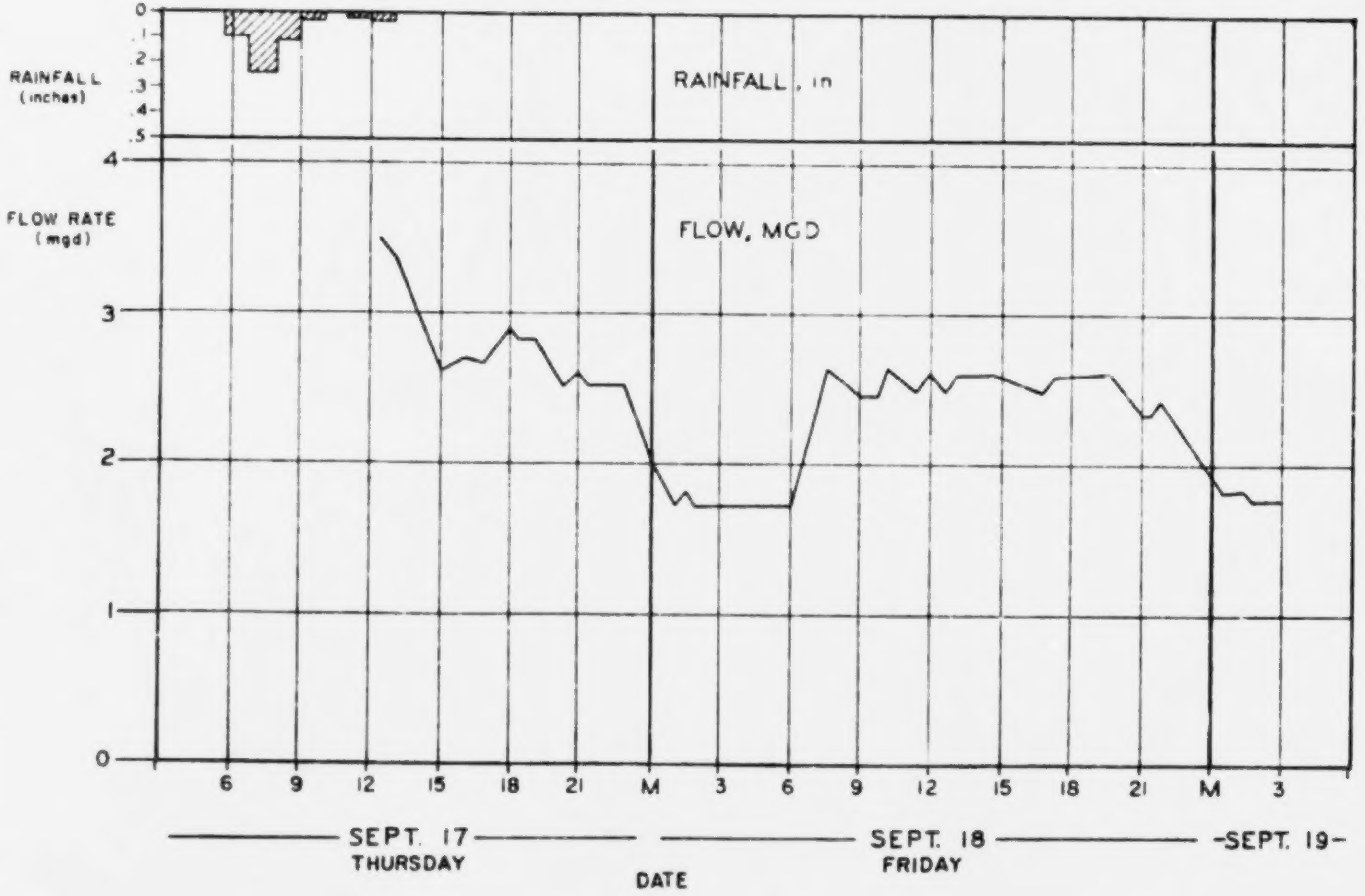


FIGURE 36 - DRY WEATHER SEWER FLOWS FOLLOWING RAINFALL FOR SEPT. 17-18 1970



The average annual precipitation is about 30 inches. About two-thirds of the annual amount occurs during the growing season. Since 1841, the wettest year was 1876 with 50.36 inches, and the driest year was 1901 with 18.69 inches. The long-term average annual snowfall is about 46 inches, but it varies considerably from season to season. During the winter of 1885-86, 109 inches were measured, while in 1884-85, the snowfall totaled only 11 inches. The 1967-68 season produced 12 inches.

Thuderstorms occur less frequently and with less severity in the Milwaukee area than in areas to the south and west. Hail size is generally 1/2 inch or less, although it has been noted as large as 2 inches in diameter with unusually severe storms. The maximum rainfall which has occurred in a 24-hour period is 5.76 inches in June, 1917. As much as 0.79 inches has fallen in 5 minutes, 1.11 inches in 10 minutes, 1.34 inches in 15 minutes, 1.86 inches in 30 minutes, and 2.25 inches in 1 hour.

Table 28 reproduces U. S. Weather bureau records tabulating monthly precipitation at the airport (Station 1) since 1932.

The storm detention tank model developed during this project is designed to operate on an input of hourly rainfall records. Punched computer cards were available from the U. S. Weather Bureau providing such data at the airport station located south of the City.

In addition to the above long term record, two rain gauges were maintained by the City of Milwaukee during the program, one in the project area, and one approximately two miles south of the area. Data from these guages were converted to punched cards using the same format as U. S. Weather Bureau data.

Table 29 compares records of the three sources of rain data, listing the volume of rain recorded each month. The pattern of variation from month to month and from year to year are similar, however distinct differences in individual months and in annual accumulations are evident. Routine differences between gauges are to be expected - even for gauges located relatively close to each other. Table 30 lists annual accumulations at two gauges maintained in the Milwaukee area by the U. S. Weather Bureau. Annual differences in the order of 10 to 15% are common.

TABLE 28
PRECIPITATION RECORD
MILWAUKEE

TOTAL PRECIPITATION

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

TOTAL SNOWFALL

Season	July	Aug.	Sept.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	Total
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

SOURCE – U.S. WEATHER BUREAU

Table 29. COMPARISON OF RAIN GAUGE RECORDS

Year	Month	INCHES OF RAIN			
		STA 1 Airport Sta. U.S. Weather Bureau	STA 4 Proj-Area Gauge	STA 2 Broadway Gauge	Inches Snow at STA 1
	J	0.98			4.6
	F	0.56			3.5
1968	M	0.31			1.2
	A	2.90		2.80	0.4
	M	3.28		2.85	
	J	7.79	6.61	5.85	
	J	3.59	3.10	3.26	
	A	2.59	2.29	3.84	
	S	3.36	2.41	3.54	
	O	0.94	1.03	1.22	
	N	2.56	2.58	2.69	0.3
	D	2.65	2.68	1.92	11.6
(Total	7 Mos.)	(23.48)	(20.70)	(22.32)	
Total	Year	31.51			

Year	Month	STA 1	STA 4	STA 2	Inches Snow at STA 1
	J	1.83	1.87	0.90	11.1
	F	0.05	0	0	0.7
1969	M	1.05	0.98	0.80	6.2
	A	3.42	3.86	4.10	
	M	3.05	3.70	3.97	
	J	7.53	5.25	6.11	
	J	6.61	4.96	3.81	
	A	0.53	0.26	0.41	
	S	2.18	0.59	0.24	
	O	4.48	3.88	5.08	
	N	1.14	0.97	1.14	0.7
	D	1.18	1.19	0.08	14.9
Total	Year	33.05	27.51	26.64	

Table 29. (Continued)

Year	Month	STA 1	STA 4	STA 2	Snow at STA 1
1970	J	0.41	0	0	6.0
	F	0.13			2.0
	M	1.62		1.09	10.7
	A	2.71	0.06	0.21	5.2
	M	3.41	4.41	3.27	
	J	3.92	2.72	3.68	
	J	1.93	1.72	1.97	
	A	0.64	0.60	0.63	
	S	6.94	6.44	7.15	
	O	2.09	1.89	2.18	
	N	2.03	1.73	1.88	0.6
	D	3.02	1.96	1.54	19.6
Total	Year	28.85	21.53	23.60	
1971	J	1.37	2.44	0.63	15.8
	F	2.50	7.23	5.72	2.5
	M	2.83	6.48	7.55	18.1
	A	1.31	5.08	0.83	0.7
	M	0.90	1.69	2.04	
	J	2.67	2.71	3.87	
	J	2.60	2.63	2.25	
	A	2.28	2.42	2.63	
	S	1.30	1.10	1.46	
	O	1.90	2.14	2.21	
	N	2.45	3.17	2.83	6.1
	D	4.34	4.30	4.76	2.7
Total	Year	26.45	42.39	36.78	

Table 29. (Continued)

Year	Month	STA 1	STA 4	STA 2	Snow at STA 1
1972	J	0.75	0.59	0.48	
	F	0.86	0.26	0.15	
	M	2.57	2.03	2.29	
	A	2.76	1.46	2.68	
	M	2.33	1.10	1.23	
	J	3.33	2.79	3.47	
	J	4.60	2.83	3.19	
	A	4.82	4.56	5.07	
	S	5.57	4.34	6.07	
	O		2.46	3.31	
Total		29.59*	19.98**	24.63**	

*First 9 months of data included

**First 10 months of data included

Table 30. COMPARISON OF ANNUAL RAINFALL AT U.S. WEATHER BUREAU STATIONS IN MILWAUKEE

Year	Rainfall (Inches) Mt. Mary College	 Milwaukee Airport
1946	33.61	
1947	33.61	
1948	26.77	
1949	26.40	
1950	32.12	
1951	36.34	36.43
1952	35.78	32.69
1953	25.55	22.87
1954	40.10	35.91
1955	26.94	27.90
1956	25.35	28.35
1957	24.32	24.95
1958	23.30	20.17
1959	32.50	37.68
1960	39.63	40.71
1961		
1962	24.89	21.91
1963	18.50	19.10
1964	31.60	28.18
1965		
1966	24.74	27.13
1967		34.02
1968		30.02
1969		33.03
1970		28.85
1971		26.45
1972		29.59 (Up to Sept.)

Annual differences between local guages and the weather bureau record at the airport are in the order of 20%, with the local (project area) gauges consistently recording less annual rainfall. The year 1971 is a notable exception, where local gauges recorded particularly high values. In this case the major variations occur early in the year and may be related to distribution of snowfall.

Hourly rainfall data for the period covered by the program are recorded in the Appendix. This listing, developed from punched cards, recording hourly rainfall at all three stations, lists hourly data on those days where precipitation occurred. Any day on which no precipitation was recorded is not listed. In addition, a synoptic listing of each storm event is presented and provides the following data on each storm event:

Date and hour of day for beginning of event
Duration (Hours)
Average Intensity (Hundredths of an inch/hour)
Total Volume (Hundredths of an inch - 0.01 inch)
Delta (Interval in hours since end of antecedent storm)
Delta Prime (Interval in hours between centroids of
storm and antecedent storm)

Station records were compared statistically, based on hourly records to perform a similar comparison with that previously made by inspection for monthly and annual totals. For the total period of record each of the local gauge records (STA 2 and STA 4) were compared individually with the weather bureau record (STA 1) and with each other. The analysis explored Total Volume, Hours of rain recorded, and average intensity for maximum hour, maximum 2 hours, and maximum 3 hours.

Based on correlation coefficients determined - the data is obviously correlated, though, as can be expected, not to very high levels. Best correlation obtained is for volume, with correlation coefficients of about 0.8. Hours of rain and intensity correlated to a lesser degree (coefficient approx. 0.[illegible]. Data from this analysis indicates the local gauges correlated with each other to a greater degree than either does with the airport gauge as expected because of the relative gauge locations. The analysis further confirms the indication that generally less rain is recorded by local stations than at the airport, based on the period of study. This may change if longer term averages were used.

The significance of these differences in recorded rainfall on storm overflow predictions cannot readily be determined from this rainfall record, since quite opposite effects are observed for individual events and even for monthly totals, than the general average conditions indicate. However, the use of long term rainfall data in projections is a valid approach for this study.

The significance of the rain gauge record selected for predicting storm overflow conditions, will be made by operating the overall model using each of the rainfall records independently and comparing the predicted output for significant differences.

SEWAGE - WET WEATHER FLOW (RUNOFF COEFFICIENT C_R)

Since the Urban Storm overflow model utilized in this project operated on hourly rainfall data, an important element of data analysis was the determination of a runoff ratio or coefficient. For a specific volume of rainfall which fell on the area during each hour of a storm event, the percentage which reached and flowed through the combined sewer system was identified. In addition, significant influences on this ratio or coefficient were important to identify. Such factors as rainfall intensity and duration, and the interval since the antecedent storm would be expected to have some effect on the observed runoff coefficient. The greater the degree to which such influences can be identified and quantified, the greater will be the accuracy of the model using such a factor in the output it predicts.

Data available from the project on rainfall and on combined sewer flows during storm events were analyzed to develop characteristics for the runoff coefficient (C_r) for the 570 acre project area. The runoff coefficient utilized in this program was defined as the ratio of the volume of storm water reaching the combined sewers, to the volume of rain which falls on the drainage area. Thus:

$$C_r = \frac{\text{Volume of runoff}}{\text{Volume of rain}}$$

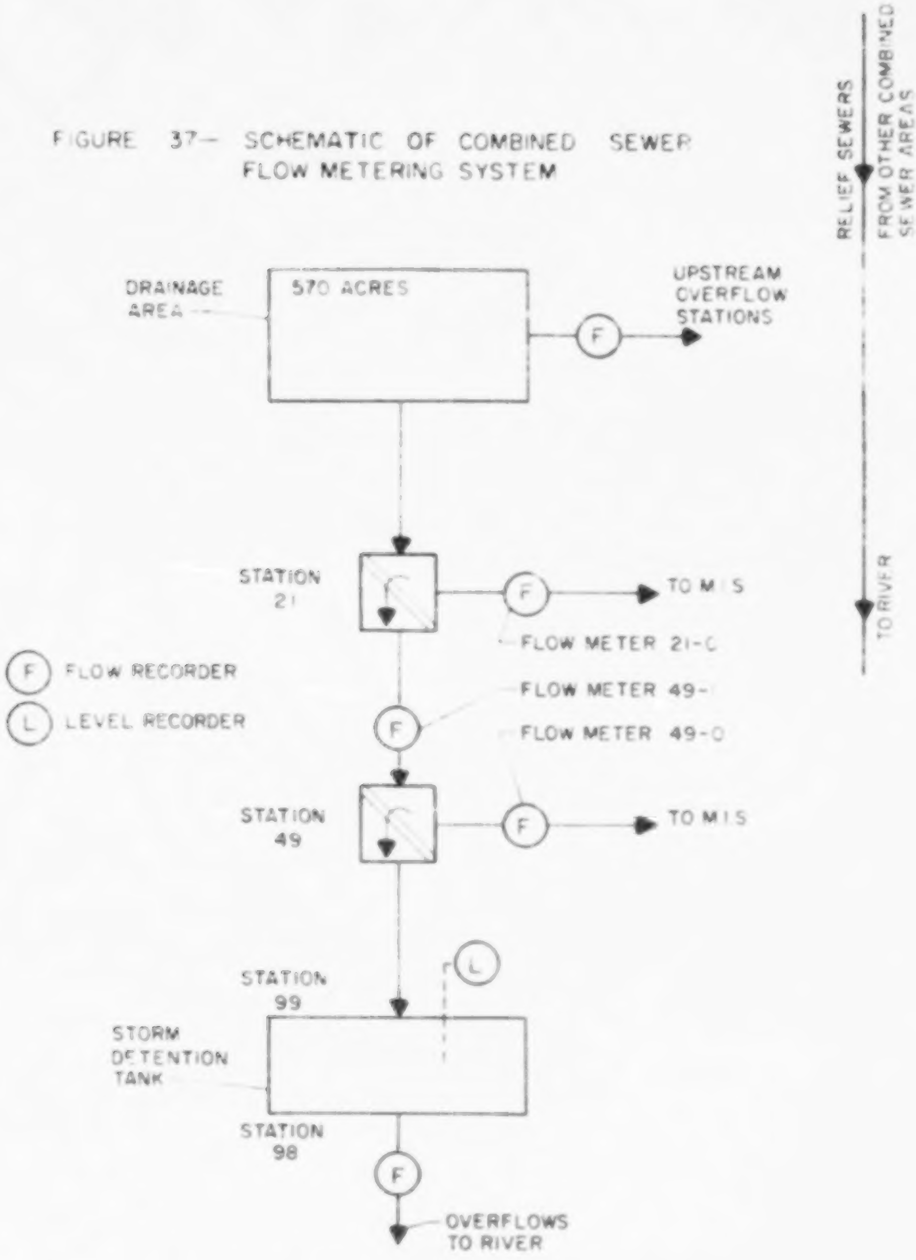
Analysis of rainfall volume consisted in taking hourly rainfall intensity data from rain gauging station No. 2 located approximately 2 miles south of the project area and station No. 4 in the project area, and computing the total volume of rain which fell on the 570 acre drainage area during each storm event. Each of the rain gauging stations was treated independently in this analysis. Data from the period November, 1971 through October, 1972 were utilized in this analysis. The analysis assumes that the rainfall recorded by a gauge represents a uniform intensity over the area. This would appear to be a reasonable assumption, based on the relatively small size of the area (less than 1 square mile). Some variation in rainfall patterns was exhibited between the two rainfall gauging stations, as would be expected. While some differences can be assumed to be due to the different gauges in use at the two stations, the gauges rather consistently record somewhat different times, intensities and volumes of rain falling, indicating a non-uniform distribution of rainfall between the gauge located in the project area and the one to the south of the area during most events. Based on this comparison, it is expected that some lesser degree of variation also may exist in the various locations throughout the project area. In the data presented, storm events on March 1, June 12, and August 14, 1972 were recorded at only one of the stations. This factor was noted, although it was not particularly significant in the operation of the predictive model. Such variations were compensated for in the overall runoff coefficient (C_R) selected for the test area. Table 31 presents a summary of the storm events utilized in the determination of C_R, and includes both rainfall recorded at the gauging station and calculated volume of rainfall.

Runoff in the combined sewers during each of these events is also presented in Table 31. Runoff volume was calculated from a flow balance using recorded flows in the combined sewer system during the storm events analyzed. Figure 37 presents a schematic representation of the combined sewer system in the test area. There are a series of "overflow stations" within the upper reaches of the project area collection system, as described in Sections IV and V of this report. These were monitored overflow stations. Experience indicated that they overflow relatively infrequently, and for quite short periods. They were treated as a single element in the analysis.

Normal dry weather flows, and combined flows from the smaller storms flow into the Metropolitan Interceptor Sewer (M. I. S) at Station 21 and were recorded (Flow meter 21-0) at that point. When flows exceed the capacity of the M. I. S. intercepting device at that station (approximately 4.5 mgd), the excess overflows and is diverted to Station 49. An inline flume in this line (Flow Meter 49-F) was installed to record flows entering Station 49. As described in Section VII, mechanical difficulties greatly reduced the amount of information obtained from this meter. Normal dry weather flows and some combined flow are diverted into the M. I. S. at Station 49 and are recorded (Flow Meter 49-0) in the same manner as at Station 21. Flows, beyond the capacity of this outlet at Station 49, overflow and enter the project detention tank.

Capacity of the M. I. S. intercepting device at Station 49 is approximately 3.0 to 3.5 mgd., resulting in a total interceptor capacity of approximately 8.0 mgd. Flow enters the M. I. S. sewer lines by passing through a submerged orifice, the capacity of which increases somewhat with higher combined sewer flows. Interceptor capacity in the study area is thus 3.5 to 4.5 times the dry weather flows.

FIGURE 37— SCHEMATIC OF COMBINED SEWER FLOW METERING SYSTEM


RELIEF SEWERS
FROM OTHER COMBINED SEWER AREAS
TO RIVER
DRAINAGE AREA
570 ACRES
UPSTREAM OVERFLOW STATIONS
F
STATION 21
F
TO M.I.S
FLOW METER 21-O
F FLOW RECORDER
L LEVEL RECORDER
F
FLOW METER 49-I
FLOW METER 49-O
STATION 49
F
TO M.I.S
L
STATION 99
STORM DETENTION TANK
STATION 98
F
OVERFLOWS TO RIVER

DATA SUMMARY OF RAINFALL - RUNOFF

Storm Date	Station 2 - Rainfall			Station 4 - Rainfall			Runoff (mm)	C_2	C_4	Storm Number
	Duration (hr)	Interval (Days)	Volume (in)	Duration (hr)	Interval (Days)	Volume (in)				
11-11-71	10	2.5	1.40	9	2.5	1.74	-	-	-	M 1
11-19-71	3	.8	.23	3	[illegible]	.21	.27	.44	.59	M 2
11-26-71	11	3.3	.41	11	3.7	.49	2.34	.37	.31	M 3
11-28-71	16	1.5	.61	20	2.5	.64	4.93	.52	.48	M 4
12-10-71	14	.8	1.34	12	.8	1.36	8.12	.54	.58	M 5
12-14-71	17	3.3	1.52	9	4.3	1.37	9.61	.41	.41	M 6
12-17-71	21	2.3	1.26	17	2.3	1.16	4.99	.29	.33	M 7
02-25-72	3	2.0	.07	2	[illegible]	.25	.55	.46	.60	M 8
03-31-72	-	-	-	7	5.0	.36	3.41	-	.61	M 9
04-21-72	9	4.2	[illegible]	11	4.2	1.06	6.90	.56	.50	M10
06-02-72	3	3.5	.41	4	1.5	.24	2.64	.42	.71	M11
06-12-72	-	-	-	5	[illegible]	.36	6.27	-	.47	M12
06-14-72	9	2.3	.62	9	2.3	.73	9.53	.78	.66	M13
06-19-72	6	4.7	[illegible]	5	4.7	.66	7.54	.43	.64	M14
07-02-72	[illegible]	.4	.41	4	.5	.52	3.88	.55	.48	M15
07-12-72	2	4.5	.60	2	5.5	.43	3.43	.33	.52	M16
07-17-72	6	3.0	.48	6	2.7	.46	3.04	.41	.43	M17
07-26-72	12	3.9	.51	-	-	-	3.69	.50	-	M18
08-02-72	7	.10	.77	7	.10	.81	7.06	.59	.56	M19
08-06-72	8	3.7	.88	7	3.3	.59	4.62	.34	.51	M20
08-11-72	8	3.7	.90	5	3.7	.83	5.62	.45	.44	M21
08-14-72	-	-	-	3	3.0	.64	5.72	-	.58	M22
08-21-72	7	5.3	.48	7	5.3	.40	3.48	.47	.56	M23
08-25-72	6	1.8	1.32	7	1.8	1.13	8.08	.40	.54	M24
09-12-72	7	1.3	1.48	9	1.6	1.02	8.12	.36	.52	M25
09-17-72	10	4.5	2.41	11	4.4	1.94	16.5	.44	.55	M26
09-28-72	5	3.0	.74	5	3.5	.60	4.81	.40	.52	M27
10-20-72	62	9.6	[illegible]	62	[illegible]	2.25	22.46	.51	.68	M28
Average								.46	.54	

Note: Run off coefficients plotted verses time since antecedent storm, duration of storm, rainfall volume, and rainfall intensity are noted in Figures 38 and 39. Figures 38 and 39 contain data points which were derived by averaging values in appropriate plotting groups and then utilizing the average values for the points.

In the original metering concept, the flows of interest in the program were to be developed as follows:

Total Q in system = $Q_{21-0} + Q_{49-F}$ + Sum of Upstream Overflows

Flow to tank = $Q_{49-F} - Q_{49-0}$

Storm runoff = Total Q - Dry weather Q

However, because of the difficulties with the flow metering facilities at Station 49-F, a flow at Station 99 was calculated based on level changes in the storm tank. The flows of interest were then developed in the following manner:

Total Q in system = $Q_{21-0} + Q_{49-0} + Q_{99}$ + Sum of Upstream Overflows

Flow to tank = Q_{99}

Storm runoff = Total Q - Dry weather Q

It is important to note that complete flow data based upon tank level could be developed in this manner without Station 49-F input, only when the detention tank was not full, wherein tank level changes could be measured. When meter 49-F continued to present operational problems creating difficulties in providing a reliable flow record, provision was also made to determine flow rates leaving the storm tank by measuring head on the outlet weir. When meter 49-F was operating comparison of the two total flows calculated for the system were made. At times when the tank filled and overflowed, storm tank flow monitoring would shift from Station 99 to Station 98 at the tank outlet.

For each specific storm event, flows recorded at all stations were summed up for the storm interval and converted to a total volume (million gallons). A listing of all recorded flow information is presented in the appendix. Data in this listing has been arranged chronologically. In calculating the runoff, where data was incomplete, some flow assumptions were made. A frequent assumption was for a flow rate of 3.5 mgd at Station 49-0 during events where overflow reaching the storm tank occurred. This assumption is based on observed values during storm events when such data was available. Dry weather flow values used in the runoff calculations are based on an average dry weather flow of 2.0 MGD.

Runoff volumes determined in this analysis are presented in Table 31, together with the rainfall data previously discussed.

In the analysis, separate rainfall volumes were computed using rain gauge stations 2 and 4. A runoff coefficient (C_r) was obtained for each set of rain data, for each storm event investigated, by calculating the ratio between the total runoff for the storm in question and the rainfall volume. Calculated values for C_r are presented in Table 11.

Runoff coefficients have been determined using a total of 28 storm events over one year of operation of the storm retention tank. Coefficient analysis was limited to those events where sufficiently complete flow information was available. The flow assumptions made and the justification for using an assumed value have been discussed above. Where flow at more than one station was missing, that event was not utilized in developing a value for C_r. Events which were utilized in assumptions are noted in Table 11, and generally yield values for C_r in the same range as for those events with complete data. In many instances all monitoring stations did not operate for a particular storm because the storm did not generate flows in excess of service sewer capacity. The amount of rainfall necessary to generate an overflow condition varies somewhat from station to station.

On the basis of the data analyzed, storm water runoff in the test area is characterized by a runoff coefficient C_r of 0.5. Observed ranges for C_r were 0.[illegible] to [illegible].

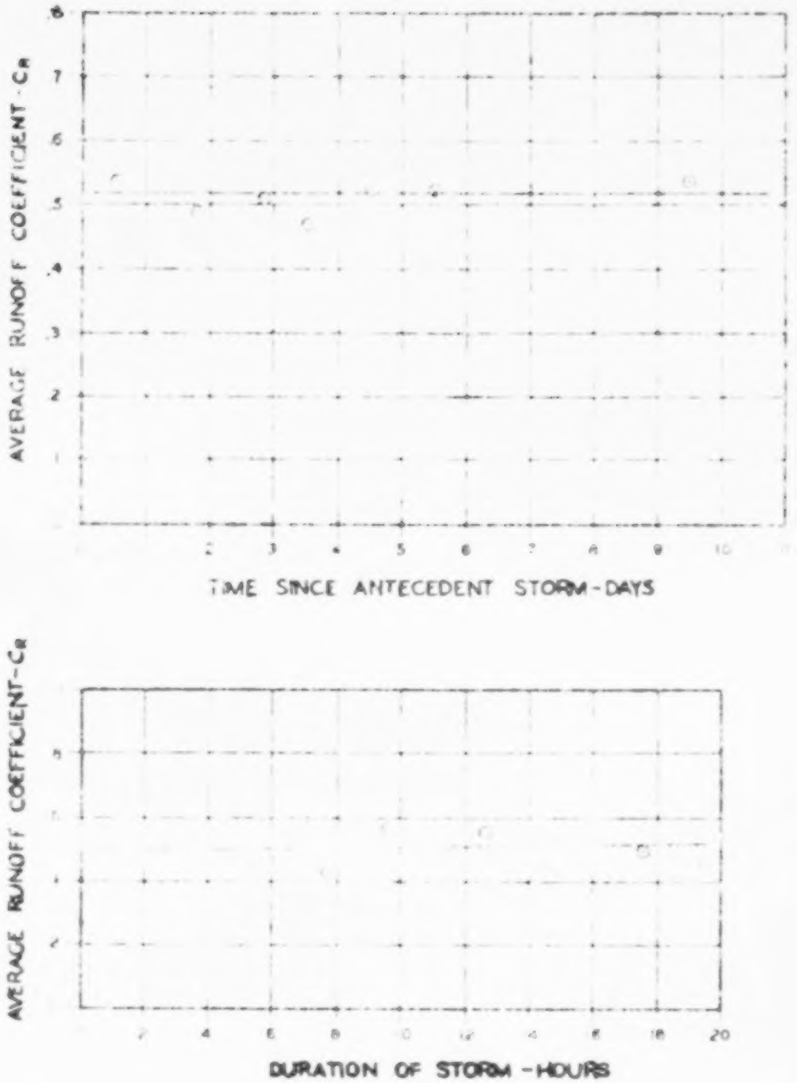
Rainfall, runoff data, and calculated C_r values were subjected to an analysis to identify the influence of various storm characteristics on the runoff coefficient. The effect of duration of the storm event, total volume of rain per storm, rainfall intensity, and interval since antecedent storm were investigated. Results of this analysis are plotted on Figures [illegible] and [illegible]. When all the storm data was evaluated, no significant variation in C_r with any of the above parameters is indicated. The data obtained was not sensitive enough to determine whether variations might exist for individual hours within a storm event.

SEWAGE - WET WEATHER QUALITY

A substantial body of basic data was secured by the sampling program for combined sewage flows during storm events. A comprehensive program of chemical analyses covering a wide range of parameters was incorporated in the test program. Sampling was done at relatively short intervals providing a large number of individual observations during each storm event, for a large number of separate events.

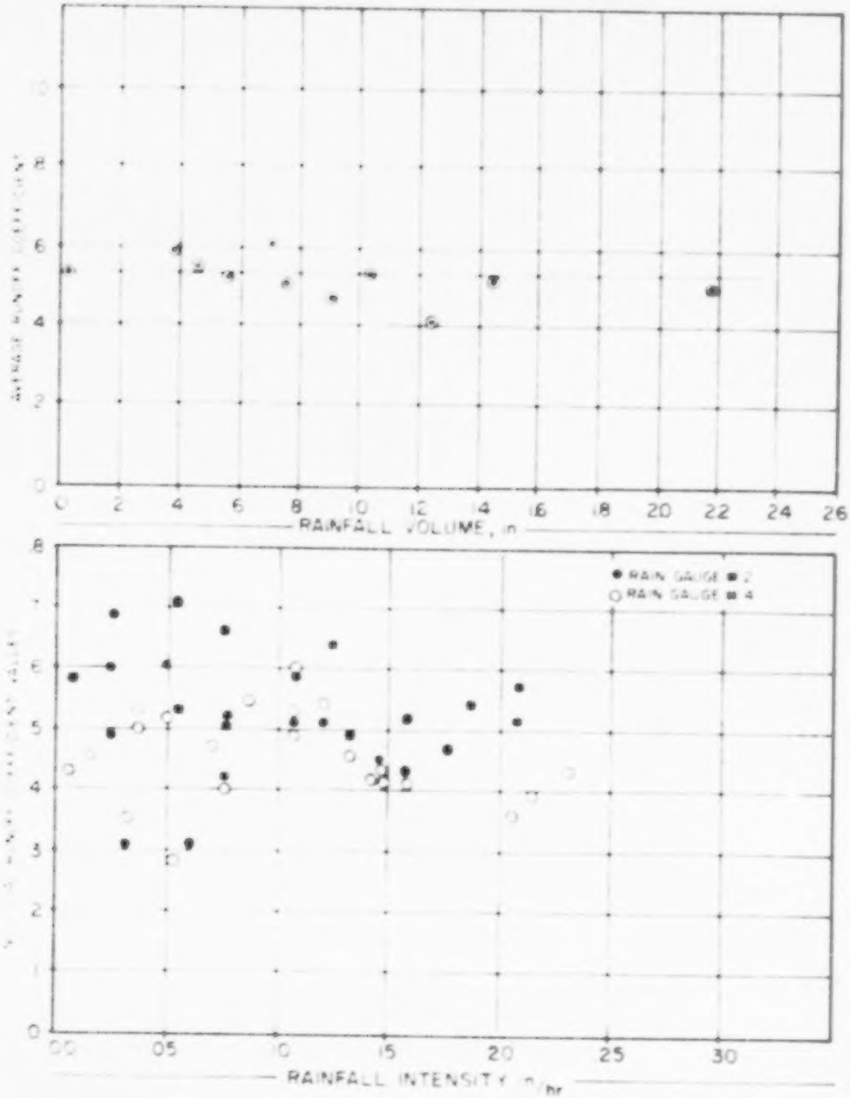
FIGURE 38 - EFFECT OF STORM CHARACTERISTICS ON RUNOFF COEFFICIENT

Table 31



Note: Data regarding individual storm characteristics was given in Table 31. Data points plotted above are based upon Table 31 information. However, the points plotted were established by plotting the average value of a plotting range. This range includes one or more individual storm data points.

FIGURE 39 — EFFECT OF STORM CHARACTERISTICS ON RUNOFF COEFFICIENT



Note: Data regarding individual storm characteristics was given in Table 31. Data points plotted above are based upon Table 31 information. However, the points plotted were established by plotting the average value of a plotting range. This range includes one or more individual storm data points.

The basic data is listed in Appendix **V,** and is organized in chronological order. Inspection of the raw data and individual plots of different events has indicated a wide variation in observed concentrations for all parameters. Many of the events, though not all, exhibit a typical "first flush" pattern. The flushing of accumulated materials on streets and in sewers and catch basins, etc., appears to result in a rapid increase in concentration of some quality parameter, to a peak value, followed by a decline to some lower value - often lower than the dry weather concentration - attributable to dilution of sewage by the relatively less concentrated storm runoff waters.

A computer program was utilized to consolidate and analyze the mass of available data. In this approach, all quality data from a storm event was sorted in accordance with elapsed time since the start of the storm event. Since automatic samplers were actuated by increased flow at the sewer monitoring station, the time associated with the initial quality sample was identified as the start of the storm event. Quality data for each storm event was analyzed individually, and the following characteristics summarized:

1. initial concentration
2. maximum concentration
3. average concentration - 0 to 30 minutes
4. average concentration - 30 to 60 minutes
5. average concentration - 1 to 2 hours
6. average concentration - all hours in excess of 2 hours

In addition, the average for all storms analyzed was determined. this data is summarized in Table 32.

The average quality variations summarized in Table 32 were based on data from 97 individual storm events. The data clearly demonstrates a pattern of quality variation which reflects the existence of a "first flush" condition. When all available data is considered on an averaged basis, every one of the quality parameters investigated demonstrates this pattern whereby concentration of the contaminant decreases with time as the storm event progresses.

When storm events are considered individually, not all events show this pattern, even though the frequency of occurrence and magnitude of the deviation from the above pattern, are not sufficient to distort the overall picture provided by analysis of a large enough sample. In many cases when deviations from the pattern occurred in an individual storm, they could be attributed to a complex storm pattern, in which a number of radical changes in rainfall intensity occured during the duration of what had been considered a single storm event. A more rigorous definition as to what constitutes a single storm event could address

TABLE [illegible]

AVERAGE OF WET WEATHER SEWER QUALITY VARIATIONS FOR ALL STORMS ANALYZED

Quality Parameter	Average Initial Value	Average Maximum Value	Average 0-30 Minute Value	Average 30-60 Minute Value	Average 1-2 Hour Value	Average Greater than Hour Value
BOD (mg/l)	150	176	143	128	97	87
SS (mg/l)	397	489	348	270	192	193
pH	6.9	7.1	6.9	6.9	6.9	6.9
[illegible] (mg/l)	[illegible]	[illegible]	436	383	280	256
[illegible]	183	210	195	131	167	11[illegible]
Nitrogen (as N)						
Kjeldahl (mg/l)	14.6	17.2	13.6	11.3	8.4	8.1
Ammonia (mg/l)	4.6	5.7	4.5	3.9	3.1	3.3
Nitrite (mg/l)	.013	.013	.013	.011	.013	.013
Nitrate (mg/l)	.827	1.005	.749	.565	.520	.624
Phosphorus as PO_4						
Ortho (mg/l)	8.8	9.4	7.6	6.1	5.4	4.9
Total (mg/l)	13.4	15.7	13.0	10.1	9.1	8.4
Total Solids (mg/l)	813	952	775	661	681	550
Total Volatile Solids (mg/l)	242	289	217	149	108	106
Coliforms						
Total no./ml	(Wide range in data, averages are not representative of data)					
Fecal no./ml	14,173	21,787	13,671	7,936	6,902	6,199

such deviations. To do so, would involve breaking down some storm events into a series of overlapping events. This analytical refinement has not been utilized in this project for several reasons. Doing so would significantly complicate what is desired to be a simple, direct, and readily useable analytical tool for identifying the effects of storm runoff in an urban area and for exploring engineering alternatives for their control. In addition, the major emphasis in the program is to identify the broader aspects of "all storm runoff" rather than one individual event, and a clearly defined pattern has been shown where all storms are considered.

Table 33 has been prepared to help provide some perspective both on the quality data for wet weather conditions and on the relationship of wet weather quality with dry weather quality. The summary in this table indicates that many parameters exhibit somewhat higher initial concentrations than average dry weather concentrations. BOD, COD, suspended and volatile suspended solids, chlorides and nitrates exhibit this effect. While the normal wide variation in dry weather quality may have some effect on this observation, the variations in the wet weather concentrations are believed to be largely due to the fact that the initial wet weather sample was in fact secured some time after the effects of the storm event had begun to be felt at the sampling station and was part of the "first flush" combined sewage concentration. The occurrence of a "first flush" phenomenon in the early stages would result in initial samples having concentrations somewhat greater than dry weather values

Table 33 lists a calculated ratio of the maximum concentration observed during storm runoff compared with the dry weather average value for the same parameter. For BOD, COD, solids, chlorides and nitrates and organic nitrogen, maximum storm values are about 1.5 to 2.5 times the dry weather average. These constituents may be assumed to be increased by virtue of materials flushed from streets, catch basins, sewer lines, etc., in the storm flow.

Several parameter initial concentrations did not increase during storm events. Ammonia, total and ortho phosphate, and fecal coliforms, all exhibited maximum concentrations less than dry weather averages (total Kjeldahl nitrogen has not been considered in itself, but rather in its individual components ammonia and organic nitrogen). This suggests the quite reasonable conclusion that the primary source of such contaminants is the sewage in the lines when the storm occurs. The observed ratios of 0.45 for ammonia and 0.58 for fecal coliform (maximum wet to dry concentration) imply a simple dilution by storm water in which they are present in significantly reduced concentrations than sewage concentrations. Phosphorus ratios are [illegible] to 0.89 showing some contribution by runoff scouring of sewer lines, but apparently less phosphorus bearing particulate matter accumulates during dry weather than does organic particulates.

TABLE 33

COMPARISON DRY AND WET WEATHER SEWAGE QUALITY

	Dry Weather Quality			Wet Weather Quality			Ratio
Parameter	Average Concentration	Range		Initial Value	Maximum Value	+ 2 Hour Value	Max DW Avg
BOD (mg/l)	112	17-323	*	150	176	87	1.57
SS (mg/l)	150	5-900	*	397	489	193	2.63
TVS (SS)(mg/l)	115	2-700	*	242	289	106	2.51
pH	7.6	6.7-8.7		6.9	7.1	6.9	
COD (mg/l)	238	35-730	*	439	581	256	2.44
Cl (mg/l)	141	3-826	*	188	210	117	1.5
TKN (mg/l-N)	20.7	5.6-47	o	14.6	17.2	8.6	0.83
Organic (mg/l-N)	9.0	4.6-16	*	10.0	11.5	5.3	1.28
NH_3 (mg/l-N)	11.7	1.0-31.2	o	4.6	5.7	3.3	0.49
NO_2 (mg/l-N)	0.01			0.01	0.01	0.01	-
NO_3 (mg/l-N)	0.5	0.1-1.6	*	0.83	1.01	0.63	2.0
Ortho-P (mg/l PO_4)	10.7	1-39	o	7.8	9.4	4.9	0.88
Total-P (mg/l PO_4)	18.4	0.5-60	o	13.4	15.7	8.4	0.85
T Col (nu/ml)	930,000						
F Col (nu/ml)	38,000	400-54000	o	14,000	22,000	6,200	0.58

*Initial Wet greater than average dry.
oInitial Wet less than average dry.

WET WEATHER QUALITY CORRELATIONS

Available data on wet weather BOD and suspended solids was subjected to a linear regression analysis to identify the degree to which characteristic concentrations are dependent on storm conditions. These parameters were selected because of their significance in the operation of the storm detention tank, and because their variation during a storm event would be comparable to variations observed with other parameters.

Individual quality data for the 97 storm events were used in this analysis. Synoptic rainfall data for storm events associated with quality records were also incorporated. The objective of this analysis was to determine the extent of a direct relationship between pertinent quality parameters and storm characteristics. For example, it is generally believed that quality of combined sewage will be influenced by the time elapsed between the current storm event and the antecedent storm. Streets, catch basins and sedimentation in the sewer lines themselves would be expected to accumulate greater quantities of contaminants over a long dry spell, and thus result in higher concentrations in the combined sewer flow during the next rainfall.

The system simulator model included provision for using varying values for runoff and quality concentrations, depending on characteristics of the storm events. The analysis performed was designed to identify such relationships if such were indicated by the data developed.

The analytical approach employed compared data for two variables and determined the degree of correlation between them using a linear regression analysis. A least squares curve fitted to the data was determined, and variance and correlation coefficients were calculated.

When quality data only is compared, high degrees of correlation were obtained for both BOD and suspended solids, between the overflow initial, maximum and zero to 30 minute average values. Correlation coefficients between 0.81 and 0.92 were obtained. High degrees of correlation were also indicated when BOD and suspended solids concentrations for samples taken more than 1 hour after the beginning of a storm were compared. Such results generally confirm the applicability of the quality variation relationships developed in the previous section.

However, attempts to correlate combined sewage quality variations with storm characteristics were unsuccessful. Comparing maximum observed BOD with rainfall intensity and duration showed no correlation. Some correlation with interval since the antecedent storm is indicated, however the degree of correlation observed is relatively small and can be neglected without introducing significant error in calculating BOD variations from storm parameters. For example, the correlation coefficient relating BOD max to interval between storms is about 0.33. An expression relating combined sewage BOD concentrations to interval between storms, would remove less than 10% of the variance between observed values and the mean value developed. The effect of such a refinement in calculating input to a storm tank based on storm data is insignificant.

This is not to say that a clear relationship between storm interval and wet weather BOD does not exist. Some dependence is in fact indicated. What the analysis does say is that, under the conditions which prevailed over the period in which the 97 events took place, whatever relationship which existed did not prove to be significant.

A comparison of suspended solids with storm parameters gave similar results.

Table 34 tabulates correlation coefficients developed by this analysis of the rainfall and quality data. From the results of this analysis, it was concluded that one is justified in characterizing combined sewer quality by the average values developed in the previous section, for various time intervals during a storm event. The data developed provides no basis for modifying such values based on characteristics of a storm.

DETENTION TANK PERFORMANCE

Where the total volume of combined sewage entering the tank did not exceed the available capacity of the tank, the entire overflow was retained, and contaminant removal was 100 percent. At times when the storm runoff volume exceeded the available tank capacity, overflow from the tank occurred. Removal of contaminants achieved under these conditions was related to the efficiency of the storm detention tank as a sedimentation device. Total removal of contaminants during an individual storm event, or over the course of a year was related to the combined effects of storage and sedimentation efficiency. In all cases, flows which left the detention tank to enter the river were chlorinated for destruction of coliform organisms. Further, any storm waters detained by the tank, were returned to the sewer system for transport to normal treatment facilities, once wet weather flows have subsided and sewer capacity was available.

A substantial portion of the overall effectiveness of a detention tank in reducing pollution reaching a water course, resides in its ability to retain storm waters and later return them to a treatment system. Detention effectiveness is determined both by the size of the tank (Storage Volume), and also by the rate at which the tank can be emptied, following a storm event. Where the emptying rate is restricted for some reason, the net effect is one of reducing the effective storage volume of the tank, since a

Table 34. WET WEATHER QUALITY CORRELATION COEFFICIENTS

Sewer BOD Initial mg/l is Correlated with.....

1	Sewer BOD 0-30 Min Avg (mg/l)	0.9000
2	Sewer BOD 30-60 Min Avg (mg/l)	0.6398
3	Sewer SS Initial (mg/l)	0.5821
4	Sewer SS Maximum (mg/l)	0.5157
5	Sewer SS 0.30 Min Avg (mg/l)	0.5877
6	Rainfall Volume 2 inches	-0.3117
7	Sewer BOD Maximum (mg/l)	0.8090

Rainfall Volume 4 Inches is Correlated with.....

1	Rainfall Volume Sta. 2 inches	0.5366

Sewer BOD 0-30 Min Avg mg/l is Correlated with.....

1	Sewer BOD Initial (mg/l)	0.9000
2	Sewer BOD 30-60 Min Avg (mg/l)	0.7021
3	Sewer SS Initial (mg/l)	0.4162
4	Sewer SS Maximum (mg/l)	0.4482
5	Sewer SS 0-30 Min Avg (mg/l)	0.5429
6	Sewer SS 30 - 60 Min Avg (mg/l)	0.3257
7	Sewer BOD Maximum (mg/l)	0.9108

Sewer BOD 30-60 Min Avg mg/l is Correlated with.....

1	Sewer BOD Initial (mg/l)	0.6398
2	Sewer BOD 0-30 Min Avg (mg/l)	0.7021
3	Sewer SS 30 - 60 Min Avg (mg/l	0.4478
4	Sewer SS Gr 2 hr Avg (mg/l)	0.3121
5	Sewer BOD Maximum (mg/l)	0.6420

Sewer BOD 1-2 Hr Avg mg/l is Correlated with.....

1	Sewer BOD Gr 2 hr Avg (mg/l)	0.6085
2	Sewer SS 30-60 Min Avg (mg/l)	0.3477
3	Sewer SS 1-2 Hr Avg (mg/l)	0.8294
4.	Sewer SS Gr 2 Hr Avg (mg/l)	0.5337

Table 34. (Continued)

Sewer BOD Gr 2 Hr Avg mg/l is Correlated with.....

5	Sewer BOD 1-2 Hr Avg (mg/l)	0.6085
11	Sewer SS 1-2 Hr Ave (mg/l)	0.5476
12	Sewer SS Gr 2 Hr Avg (mg/l)	0.9244

Sewer SS Initial mg/l is Correlated with.....

1	Sewer BOD Initial (mg/l)	0.5821
3	Sewer BOD 0-30 Min Avg (mg/l)	0.4162
8	Sewer SS Maximum (mg/l)	0.9227
9	Sewer SS 0-30 Min Avg (mg/l)	0.9054
10	Sewer SS 30-60 Min Avg (mg/l)	0.4284
14	Sewer BOD Maximum (mg/l)	0.4407

Sewer SS Maximum mg/l is Correlated with.....

1	Sewer BOD Initial (mg/l)	0.5157
3	Sewer BOD 0-30 Min Avg (mg/l)	0.4432
7	Sewer SS Initial (mg/l)	0.9227
9	Sewer SS 0-30 Min Avg (mg/l)	0.9368
10	Sewer SS 30-60 Min Avg (mg/l)	0.5996
14	Sewer BOD Maximum (mg/l)	0.5460

Sewer SS 0-30 Min Avg mg/l is Correlated with.....

1	Sewer BOD Initial (mg/l)	0.5877
3	Sewer BOD 0-30 Min Avg (mg/l)	0.5429
7	Sewer SS Initial (mg/l)	0.9054
8	Sewer SS Maximum (mg/l)	0.9368
10	Sewer SS 30-60 Min Avg (mg/l)	0.6285
14	Sewer BOD Maximum (mg/l)	0.5466

Sewer SS 30 - 60 Min Avg mg/l is Correlated with.....

3	Sewer BOD 0-30 Min Avg (mg/l)	0.3257
4	Sewer BOD 30-60 Min Avg (mg/l)	0.4478
5	Sewer BOD 1-2 Hr Avg (mg/l)	0.3477
7	Sewer SS Initial (mg/l)	0.4284
8	Sewer SS Maximum (mg/l)	0.5996
9	Sewer SS 0-30 Min. Avg (mg/l)	0.6285
11	Sewer SS 1-2 Hr Avg (mg/l)	0.5894
12	Sewer SS Gr 2 Hr Avg (mg/l)	0.3647
14	Sewer BOD Maximum (mg/l)	0.3441

Table 34. (Continued)

Sewer SS 1-2 Hr Avg mg/l is Correlated with....

5	Sewer BOD 1-2 Hr Avg (mg/l)	0.8294
6	Sewer BOD Gr 2 Hr Avg (mg/l)	0.5476
10	Sewer SS 30-60 Min Avg (mg/l)	0.5894
12	Sewer SS Gr 2 Hr Avg (mg/l)	0.5142

Sewer SS Gr 2 Hr Avg mg/l is Correlated with.....

4	Sewer BOD 30-60 Min Avg (mg/l)	0.3121
5	Sewer BOD 1-2 Hr Avg (mg/l)	0.5337
6	Sewer BOD Gr 2 Hr Avg (mg/l)	0.9244
10	Sewer SS 30-60 Min Avg (mg/l)	0.3647
11	Sewer SS 1-2 Hr Avg (mg/l)	0.5142

Rainfall Volume 2 Inches is Correlated with.....

1	Sewer BOD Initial (mg/l)	-0.3117
2	Rainfall Volume St. 4 Inches	0.5366

Sewer BOD Maximum mg/l is Correlated with.....

1	Sewer BOD Initial (mg/l)	0.8090
3	Sewer BOD 0-30 Min Avg (mg/l)	0.9108
4	Sewer BOD 30-60 Min Avg (mg/l)	0.6420
7	Sewer SS Initial (mg/l)	0.4407
8	Sewer SS Maximum (mg/l)	0.5460
9	Sewer SS 0-30 Min Avg (mg/l)	0.5466
10	Sewer SS 30-60 Min Avg (mg/l)	0.3441

Rainfall Duration St. 2 Hrs is Correlated with.....

4	Rainfall Duration St. 4 (hr)	0.6006
5	Rainfall Intensity St. 4 (in/hr)	0.4249
7	Rainfall Volume St. 4 (in)	0.4657
8	Rainfall Volume St. 2 (in)	0.6156

Rainfall Intensity St. 2 in/hr is Correlated with.....

3	Rainfall Delta St. 2(hr)	-0.3586
5	Rainfall Intensity St. 4 (in/hr)	0.3109
8	Rainfall Volume St. 2 (in)	0.8117

Table [illegible]. (Continued)

Rainfall Delta St. 2 hrs is Correlated with.....

2	Rainfall Intensity St. 2 (in/hr)	-0.35[illegible]
6	Rainfall Delta St. 4 (hr)	0.4293
9	Sewer BOD Maximum (mg/l)	0.3240

Rainfall Duration St. 4 hrs is Correlated with.....

1	Rainfall Duration St. 2 (hr)	0.6006
5	Rainfall Intensity St. 4 (in/hr)	0.3695
7	Rainfall Volume St. 4 (in)	0.4865
8	Rainfall Volume St. 2 (in)	0.6164

Rainfall Intensity St. 4 in/hr is Correlated with.....

1	Rainfall Duration St. 2 (hr)	0.4249
2	Rainfall Intensity St. 2 (in/hr)	0.3109
4	Rainfall Duration St. 4 (hr)	0.3695
7	Rainfall Volume St. 4 (in)	0.9567
8	Rainfall Volume St. 2 (in)	0.5707

Rainfall Delta St. 4 hrs is Correlated with.....

3	Rainfall Delta St. 2 (hr)	0.4293
9	Sewer BOD Maximum (mg/l)	0.3323

Rainfall Volume St. 4 in is Correlated with.....

1	Rainfall Duration St. 2 (hr)	0.4657
4	Rainfall Duration St. 4 (hr)	0.4865
5	Rainfall Intensity St. 4 (in/hr)	0.9567
8	Rainfall Volume St. 2 (in)	0.5360

Rainfall Volume St. 2 in is Correlated with.....

1	Rainfall Duration St. 2 (hr)	0.[illegible]
2	Rainfall Intensity St. 2 (in/hr)	0.6117
4	Rainfall Duration St. 4 (hr)	0.6164
5	Rainfall Intensity St. 4 (in/hr)	0.5707
7	Rainfall Volume St. 4 (in)	0.5360

Sewer BOD Maximum mg/l is Correlated with.....

3	Rainfall Delta St. 2 (hr)	0.3240
6	Rainfall Delta St. 4 (hr)	0.3323

Table 34. (Continued)

SS Max. mg/l is correlated with.....

1 Rainfall Duration St. 2	0.355
2 Rainfall Intensity St. 2	0.194
3 Rainfall Delta St. 2	0.004
4 Rainfall Duration St. 4	0.112
5 Rainfall Intensity St. 4	0.016
6 Rainfall Delta St. 4	-0.003
7 Rainfall Volume St. 4	-0.084
8 Rainfall Volume St. 4	0.273

BOD Max. mg/l is correlated with.....

1 Rainfall Duration St. 2	-0.184
2 Rainfall Intensity St. 2	-0.121
3 Rainfall Delta St. 2	0.324
4 Rainfall Duration St. 4	-0.277
5 Rainfall Intensity St. 4	-0.288
6 Rainfall Delta St. 4	0.332
7 Rainfall Volume St. 4	-0.297
8 Rainfall Volume St. 2	-0.227

subsequent storm will be more likely to occur with some volume from the previous one still in the tank.

Evaluation of the overall capabilities of the detention tank must therefore take into account both capability for retaining overflows, and capability for removing contaminants by sedimentation in a flow-through situation. The former element can be handled readily by simple counting procedures which account for tank levels and overflow volumes at regular intervals of time.

The latter factor, sedimentation efficiency, under conditions which exist in a storm detention tank, presents a more difficult situation. The basic approach to defining removal of settleable contaminants by sedimentation is straightforward, and many examples are available in the literature. For example, Fair and Geyer[illegible] present typical settling curves for BOD and suspended solids in sewage. Removals due to simple sedimentation can be expressed by an equation of the form:

$$\text{Percent removal} = a(1 - e^{-kt})$$

In this expression the constant "a" reflects the fraction of the insoluble BOD or solids which will be removable by plain settling; the constant k defines the rate at which the contaminants settle. As the expression indicates, removals are related directly to sedimentation time (t), or in the case at hand, the amount of time a particular "batch" of storm runoff resides in the tank until it is displaced out the overflow end of the tank.

The analysis of the storm detention time as a sedimentation device presents a quite complex situation. Flows entering the tank vary erratically in response to fluctions in rainfall from hour to hour during a storm. Detention time, as a result, is variable. In addition contaminant concentrations are subject to fluctuations each hour of the storm. One might therefore anticipate quite different performance results for a storm pattern which fills the tank slowly and then surges due to high rainfall intensity occurring just as the tank fills, compared with a storm pattern in which high flow surges occur when the tank still has reserve capacity.

In order to evaluate performance efficiency of the storm tank during tank overflow occurrences, both flow and quality data at influent and overflow were necessary over the duration of a storm. Table 35 summarizes the periods when data relating to tank overflow was available. Where quality data was present, an overflow from the detention tank was indicated. At such times, i.e., when tank is full and overflowing, influent flow will be approximately equal to the overflow rate. The table indicates that complete data for the detention tank was available during a series of events in September, 1972. Continued malfunction of the flow meter at Station 49-F, which was to have provided required flow data on the tank prevented the accumulation of necessary data from this source. However, a backup source of flow data was available by converting changes in liquid level in the detention tank reflected by a level recorder to flow rates. Flow data generated from this information source was designated Station 99.

Much useful data was developed using this alternative; however, input for evaluating sedimentation efficiency during overflow from the tank could not be developed since the flow record ended when overflow began. Flow information during overflow events was recorded when installation of a supplemental metering device on the tank effluent (Station 98) was completed. In Table 36, flow data determined from this source are reported as flows from Station 99 (flow into tank) for simplicity, although the source of flow data shifted as the overflow began.

There are five listed dates for which both inlet and outlet flow and quality are available. However, in the data analysis program, the events of September 20 and 21, 1972 were considered as a single storm event. Raw data on BOD and suspended solids and on flows from these four events is tabulated in Table 36. Inspection indicates several gaps in the data on these four events. Specifically during the event of September 17 - 18, flow and influent BOD records terminate between 4 and [illegible] A. M., due to equipment malfunction, well before the end of the storm. Influent suspended solids data is incomplete after 4 A. M. In this case, for analysis of performance, only the first segment of the storm, for which complete data existed, was considered. During the event of September 20 - 21, 1972, influent BOD and suspended solids record end prematurely due to equipment malfunction or sample handling or transport problems. In this case, concentrations were assumed for the missing values near the end of the event.

In the overall model describing storm runoff and detention tank performance, data input is in the form of hourly rainfall, and contaminant concentrations based on storm parameters. To evaluate the accuracy of the component of this model which describes tank performance, the detention tank model was modified to accept observed flows and contaminant concentrations at the tank inlet, and to calculate the time variable contaminant loading leaving the tank.

Input data for this verification analysis was developed from the raw data listed in Table 36. In order to provide a continuous set of influent conditions over the period of the storm event, influent flow and concentrations were interpolated and/or averaged to provide hourly input values. Table 37 summarizes translation of actual influent data to hourly averages used as input in the storm tank model. In this table, actual data on tank effluent BOD and suspended solids is also shown. From recorded flow and

Table 35. COMPARISON OF DATA RECORDS FOR EVALUATION OF DETENTION TANK PERFORMANCE
(Hours During Day for Which Indicated Data is Available)

Date		Quality Data at Station 98	Flow at Station 99
11-01-71		1800-2400	1515-1645
11-02-71		0100-0500	None
11-28-71		0945-1745	2200 (11/28) - 0330 (11/29)
12-10-71		0630-1500	0200-0700
12-15-71		0320-1000	2330 (12/14) - 0330 (12/15)
03-07-72		1520-2330	0030-1500
03-16-72		1130-2330	0030-1000
03-17-72		0330-1620	None
03-18-72		1100-2300	None
03-20-72		1630-1730	0900-1500
03-21-72		1200-1300	None
04-20-72		2330	None
04-21-72		0130	1200-2330
06-14-72		1330-2130	0630-0730
07-14-72		2145-2300	None
07-15-72		0100	None
08-25-72		2400	2000-2300
08-26-72		0100	None
09-13-72	(X)	0150-0545	2330 (09/12) - 0730 (09/13)
09-18-72	(X)	0240-2400	1745 (09/17) - 0615 (09/18)
09-19-72	(X)	1230-1430	1000-1500
09-20-72	(X)	1700-2300	2045-2345
09-21-72	(X)	0100-0700	0015-0845
10-22-72		1100-2000	0630-2400
10-23-72		1200-1800	0030-1500

Flow at Station 99 reflects flow into tank (and flow from tank when it is full and overflowing).
Dates on which tank overflow data coincide with quality data.

(X) Dates on which influent quality data (Station 49) coincided with influent flow data.

TABLE 3[illegible]

SUMMARY OF RAW DATA FOR TANK PERFORMANCE ANALYSIS

Date	Time	Rainfall (in/hr.) 2	Rainfall (in/hr.) 4	BOD (mg/l) Influent	BOD (mg/l) Overflow	SS (mg/l) Influent	SS (mg/l) Overflow	Flow (mgd est.)
9-12-72	2330							7.1
	2345			[illegible]		370		78.0
9-13-72	0000	.24	.12	66.0		440		65.2
	0015			110.7		221		
	0030			31.3		168		
	0045			36.5		226		
	0100	.94	.58	40.5		139		34.4
	0115			59.5		222		
	0130			77.3		472		7.6
	0145			31.5	196.6	86	480	
	0200	.14	.03					1.0
	0230							.1
	0300	.02	.02					0
	0330			26.0		54		7.3
	0345			40.5		109		
	0400	.06	.10	29.0	288.0	126		2.1
	0430							1.0
	0500	.06	.01					.1
	0530				114.2		298	13.2
	0600		.12					6.5
	0630			121.0		331		1.9
	0700		.03					1.7
	0730			129.5		265		.3
9-17-72	1745			409.7				14.2
	1800	.12	.06					
	1815							2.8
	1900	.03	.01					0
	1915							0
	1945							1.4
	2000	.04	.03					
	2015			101.5		116		4.2
	2045			51.5		16		2.8
	2100	.09	.05					
	2115							5.7
	2145							0

TABLE 36
(continued)

SUMMARY OF RAW DATA FOR TANK PERFORMANCE ANALYSIS

Dates	Time	Rainfall (in/hr.) 2	Rainfall (in/hr.) 4	BOD (mg/l) Influent	BOD (mg/l) Overflow	SS (mg/l) Influent	SS (mg/l) Overflow	Flow (MGD at t)
9-17-72	2215							0
	2245							1.4
	2315							0
	2345							0
	2400	.00	.05					
9-18-72	0015							0
	0030			71.5		84		0
	0045			107.0		187		0
	0100	.00	.01					0
	0115			85.2		92		96.4
	0130			54.8		383		
	0145			35.2		268		83.8
	0200	1.4	1.04	37.8		275		
	0215			69.0		174		93.4
	0230							
	0245			40.0	35.1	134	146	47.8
	0300	.68	.56	60.0		91		
	0315			74.0		136		27.3
	0330			77.0		107		
	0345			81.5	94.1	160	320	15.1
	0400	.02	.10					
	0415							8.2
	0430							
	0445				118.6		370	8.2
	0500	.00	.01					
	0515							8.2
	0530							
	0545				86.1		332	8.2
	0600	.02	.03					
	0615							5.3
	0630							
	0645				120.6		306	
	0700							
	0715							
	0730							

TABLE [illegible]
(continued)

SUMMARY OF RAW DATA FOR TANK PERFORMANCE ANALYSIS

Dates	Time	Rainfall (in/hr.) 2	Rainfall (in/hr.) 4	BOD (mg/l) Influent	BOD (mg/l) Overflow	SS (mg/l) Influent	SS (mg/l) Overflow	Flow (MGD at t)
9-18-72	0745				26.8		139	
	0800							
	0815							
	0830							
	0845				74.6		119	
	0900							
	0915							
	0930							
	0945				32.6		99	
	1000							
	1015							
	1030							
	1045				31.1		163	
	1100							
	1115							
	1130							
	1145				53.6		194	
	1200							
	1240				18.8		139	
	1340				65.1		191	
	1440				100.6		177	
	1540				17.6		56	
	1700			119.0	27.6	694	82	
	1755			200.0	62.6	172	276	
	1850			116.5	19.6	182	66	
	1945			127.8	23.5	79	75	
	2040			99.0	43.1	60	158	
	2135			101.5	47.1	73	155	
	2325			70.2	20.1	105	22	
	2400				37.4		104	
9-19-72	1000			180.3		211		9.9
	1030							5.7
	1045			106				
	1100			99		122		0

TABLE [illegible]
(continued)

SUMMARY OF RAW DATA FOR TANK
PERFORMANCE ANALYSIS

Dates	Time	Rainfall (in/hr.) 2	4	BOD (mg/l) Influent	Overflow	SS (mg/l) Influent	Overflow	Flow (MGD at t)
9-19-72	1130			54.0		127		5.7
	1145			64.0		97		
	1200			57.0		93		3.9
	1230			60.7	22.8	80	97	2.1
	1245			69.0		125		
	1300			114.5				4.3
	1330				37.3			1.7
	1400							.6
	1430				23.8			.4
	1500							.1
9-20-72	1700				55.0		118	
	1800				6.3		93	
	1900				7.5		66	
	2000				12.5		89	
	2045				10.0	215	86	26.3
	2100			20.0]				
	2115			20.0]				43.1
	2140			20.0]				
	2145				11.3	166	110	78.4
	2200			23.8]				
	2215			23.8]				34.9
	2240			23.8]				
	2245				10.0	102	106	20.0
	2300			16.7]				
	2315			16.7]				0
	2340			16.7]				
	2345							11.5
9-21-72	0000			22.5]		12.7		
	0015			22.5]				0
	0040			22.5]				
	0045							[illegible]
	0100			12.5]	16.0	115	138	
	0115			12.5]				29.7
	0145			12.5]				24.3
	0200			12.5]	7.5		115	

TABLE 36
(continued)

SUMMARY OF RAW DATA FOR TANK
PERFORMANCE ANALYSIS

Dates	Time	Rainfall (in/hr.) 2	4	BOD (mg/l) Influent	Overflow	SS (mg/l) Influent	Overflow	Flow (MGD at t)
9-21-72	0215							20.0
	0245							11.5
	0300				16.5		161	
	0315							15.1
	0345							20.0
	0400				13.0		71	
	0415							11.5
	0445							5.3
	0500				11.2		59	
	0515							5.3
	0545							5.3
	0600				22.5		82	
	0615							5.3
	0645							5.3
	0700				15.0			
	0715							5.3
	0745							5.3
	0815							5.3
	0845							5.3

Composite Sample.

TABLE 37

AVERAGED TANK INFLUENT AND OBSERVED TANK OVERFLOW

Date	Hour	Flow (mg/hr)	BOD in mg/l	BOD in lbs/hr	BOD out mg/l	BOD out lbs/hr	SS in mg/l	SS in lbs/hr	SS out mg/l	SS out lbs/hr
9/12	24	1.772	140	2070			300			
9/13	1	2.075	60	1038			220			
	2	.179	50	74	150	224	225		478	715
	3	.002	10	0			0			
	4	.195	35	57	200	325	110			
	5	.022	0	0			140			
	6	.410	60	205	114	390	200		305	1050
	7	.075	120	75			300			
	8	.006	130	6			300			
9/17	18	.295	300	740						
	19	.058	260	126						
	20	.029	150	36						
	21	.145	80	97			70			
	22	.118	50	49			75			
	23	.029	40	9			80			
	24	.029	40	9			85			
9/18	1	.010	90	7			130			
	2	3.75	60	1876			230			
	3	2.937	55	1347	50	1230	180		145	3570
	4	.883	70	515	78	575	125		320	2360
	5	.341	60	170	110	312	120		370	1050
	6	.341	50	142	86	244	110		320	910
	7	.110	40	36	99	91	100		300	276
9/19	10	.206	90	154			180			
	11	.118	140	138			160			
	12	.199	70	116			115			
	13	.133	80	88			105			
	14	.047	10	3			95			
	15	.010	0	0						
9/20	22	2.531	20	422	7	141	180		110	1550
	23	1.143	23.8	227	11	107	120		100	1000
	24	.239	16.7	33	10	20	60			
9/21	1	.416	22.5	79	16	55	60		140	487
	2	1.125	25	121	7	70	100		115	1080
	3	.656	25	136	16	90	100		150	825
	4	.731	25	152	13	79	100		70	428
	5	.350	25	72	11	33	100		60	175
	6	.220	25	46	22	41	100		80	146
	7	.220	25	46	15	27	100			
	8	.220	25	46	15	27	100			
	9	.220	25	46	15	27	100			

concentration values the total load in pounds per hour leaving the tank was calculated and is listed.

The detention tank performance with regard to BOD and suspended solids removal is calculated by the tank model on an hour-by-hour basis. The program assigns a plug-flow pattern to storm waters passing through the tank and tracks each hourly input individually. Removals by sedimentation are assigned to each hourly input on the basis of time of detention in the tank for that batch of storm water at the point when it is displaced from the tank. A variety of coefficients for the equation describing sedimentation efficiency were investigated. All provide generally similar results, but vary the magnitude of the predicted hourly value to some degree. The removal equations selected for use in the model on the basis of comparison of observed versus predicted tank discharge are:

$$\%\ \text{Removal (BOD)} = 25\,(1 - e^{-0.20t})$$

$$\%\ \text{Removal (Suspended Solids)} = 40\,(1 - e^{-0.20t})$$

Table 38 compares actual and predicted discharges of BOD and suspended solids from the detention tank for the four storm events which have been analyzed. Additional storm data which could be analyzed would permit further refinements; however, a comparison of predicted with actual values indicates that the model developed does effectively account for but does not duplicate the relatively complex flow and load fluctuations.

TABLE 38

DETENTION TANK PERFORMANCE - ACTUAL VS PREDICTED TANK DISCHARGE (Lbs/Hr)

Date	Hour	Flow (mg/hr)	BOD Actual	BOD Model* Prediction	SS Actual	SS Model Prediction
9/12	24	0	0	0	0	0
9/13	1	.312		403		676
	2	.179	224	225	715	380
	3	.002		2		3
	4	.195	325	197		382
	5	.022		22		42
	6	.410	390	395	1050	739
	7	.074		71		131
	8	.006		5		10
9/17	18	0		0		0
	19	0		0		0
	20	0		0		0
	21	0		0		0
	22	0		0		0
	23	0		0		0
	24	0		0		0
9/18	1	0		0		0
	2	1.549		1158		986
	3	2.937	1230	1384	3570	4905
	4	.883	575	405	2360	1470
	5	.341	312	147	1050	493
	6	.341	244	139	910	420
	7	.110	91	43	276	129
9/19	10	0		0		0
	11	0		0		0
	12	0		0		0
	13	0		0		0
	14	0		0		0
9/20	22	1.692	141	666	1550	1268
	23	1.143	107	496	1000	742
	24	.239	20	111		132
9/21	1	.416	55	65	487	444
	2	1.125	70	161	1080	1316
	3	.656	90	92	875	736
	4	.731	79	109	425	686
	5	.349	33	57	175	252
	6	.220	41	35	146	154
	7	.220	27	35		150
	8	.220	27	26		88
	9	.220	27	31		74

*Using % Removal (BOD) = 25 $(1-e^{-.20t})$ % Removal (Solids) = 40 $(1-e^{-0.20t})$

SECTION IX

STORM OVERFLOW MODELING

STORM DETENTION TANK MODEL DESCRIPTION

The purpose and value of a mathematical model is that it provides a basis for (1) predicting effects under conditions other than those directly encountered during the test program; (2) predicting these effects with a relatively high confidence level, in lieu of "seat-of-the-pants" judgments, which in situations with complex interactions may be either highly speculative or impossible to make; and (3) making such predictions which can be accomplished simply, accurately and rapidly allowing a wide range of individual alternatives to be explored.

The system model which has been developed utilizing information during the data collection period of this project makes it possible to evaluate the quantity of storm water and pollutants resulting from storm overflows, which can be intercepted by a storm detention tank. The model will have a general value in that evaluations may be made for tanks of various size, serving a range of drainage areas, and a variety of rainfall conditions.

The model which has been developed is a system simulator which takes an input of hourly rainfall data. It calculates combined sewage flow and quality on the basis of characteristics of the storm event. Elements utilized in the calculation include: time of day, dry weather flow, time since start of current storm event, drainage area, and interceptor capacity. Provision had originally been made for modifying both runoff and quality on the basis of storm characteristics (Intensity, duration, interval since antecedent storm), however analysis of data obtained in the program indicates that such a refinement is not practical.

Hourly rainfall data is the basic input to the model. Calculations are made each hour of a storm event, whereby rainfall recorded on the drainage area is converted to a runoff volume. This storm runoff is combined with dry weather flow in the combined sewer system and total sewer flow determined.

Based on the interceptor capacity assigned by the program in relation to total sewer flow, overflow volume either discharged to the receiving water or entering the detention tank is determined on an hourly basis. Concentrations of quality parameters of interest are assigned to the combined sewer flow on the basis of time elapsed since the start of the storm event. The program calculates for each hour of the event the pollutant load (pounds) in the storm overflow from flow and concentration values.

When the detention tank is incorporated, the model program accounts for the amount of flow and contaminant load which enters the tank each hour. When available volume of the tank is exceeded by the overflow volume, the program calculates on an hourly basis the amount of flow and contaminant leaving the detention tank and entering the receiving water. The program accounts for both what is retained by the tank and later returned to the sewer system, and what is removed through sedimentation during those periods when overflow from the tank occurs.

Analysis of model predictions verses observed quality variations in overflows leaving the tank has verified the general validity of the sedimentation model employed in the program. This model assigns a plug flow pattern to combined sewage entering the tank and maintains an inventory of each hour's input. Removals by sedimentation are calculated using the relationships developed, based on the amount of time each hour of input has taken to pass through the tank to the point when it overflows and leaves the tank.

The model thus determines both flow and pounds of contaminant (BOD and suspended solids) at hourly intervals in the overflow from the combined sewer system, which is discharged to the river. This time variable loading serves as input for a water quality model of the receiving water. The effect of a specific storm overflow can then be projected as an impact on river water quality.

The output format of the model has been arranged so that a summary of each storm event is listed hour by hour, and totals for the particular event are listed. This listing includes flow and pounds of BOD and suspended solids overflowing from the collection system and entering the detention tank, as well as similar data on whatever leaves the tank to enter the receiving water.

The input format is arranged such that a variety of engineering alternatives may be tested readily, in evaluating overall performance under various conditions subject to either variation from one location to another, or to control by engineering design. For example, the following parameters defined by the characteristics of the location to be investigated may be readily adjusted in execution of the model: (a) drainage area, (b) capacity of interceptor sewers; (c) dry weather sewer flow, (d) sewage quality, both dry weather and wet weather conditions, (e) runoff coefficient. Important elements which are subject to definition or modification by engineering design may also be readily adjusted to permit measurement of their effect. Examples in this category are - (a) size of storm detention tank relative to the area to be served, (b) interceptor capacity, (c) rate at which tank is pumped out at the end of a storm event.

A Fortran description of the model, together with output summaries of a series of runs is presented in the Appendix.

MODEL COEFFICIENTS

As discussed previously the storm system model utilizes a series of variable inputs to reflect specific characteristics of the geographical area being investigated, and of the quality variations which may either be general in nature or specific to the area in question. These parameters, which have been utilized in the analysis of detention tank effectiveness, are listed below and discussed briefly. The data supporting their selection has been presented in Section VIII.

1. Drainage Area - The project area in Milwaukee from which data was obtained, covers approximately 570 acres (0.9 square mile). A brief discussion of the project area is included in Section III.

2. Dry Weather Flow - An average daily flow of 2 MGD has been selected, however model input is based on hourly values over a 24 hour morning hours, to 2.3 MGD between 8 A.M. and 10 P.M. These minimum and maximum flows are the average values established per a typical day, as indicated on Figure 35. However, on specific days hourly rates of as low as 0.9 MGD and as high as 2.9 MGD have been observed.

3. Rainfall - Hourly rainfall records from a local rain gauge which accurately reflects precipitation in the area to be investigated, are used. Records from both local rain gauges in and adjacent to the test area and from a U.S. Weather Bureau Station located several miles to the south of the test area have been used in the analysis performed in this study.

4. Runoff Coefficient - A runoff coefficient of 0.5 has been utilized in all program runs because of the great number of variables attendant to natural phenomenon. The value of this coefficient is maintained at a constant for practical reasons, and is not modified in the program

with variations in storm characteristics. This coefficient reflects the fraction of the total rainfall recorded which reaches and flows through the sewer system. It may be considered to account for only losses due to infiltration or percolation into the ground, ponding and subsequent evaporation, but also to average out discrepancies, which would be caused by variations in intensities actually encountered over the test area as a whole compared with what occurs at the rain gauge.

5. Interceptor Capacity - Interceptor capacity was selected as 8.0 MGD for the project area, or approximately 4 times dry weather flow. This reflects the amount of flow including runoff from storms, which can be retained in the sewer system without overflowing at relief points.

6. Detention Tank Volume - The detention tank installed at the Humboldt Avenue location, has a capacity of approximately 3.9 million gallons.

7. Tank Emptying Rate - The pump out rate actually experienced during the test program has proved to be quite variable. To empty the tank following a storm event, tank contents are pumped back into the interceptor and will ultimately reach the wastewater treatment plant. The sewer line into which the tank contents are pumped frequently remained surcharged for a significant period. The tank discharge pumps were regulated from a liquid level in this sewer line, so that during the program the emptying rate was quite variable. Several different emptying rates were accordingly explored during program execution (24 hr.,48 hr. and 96 hrs.)

8. Wet Weather Sewage Quality - Sewage quality and variations during combined storm and sanitary sewage flows, have been characterized by analysis of data from some 97 storm events. Detention tank effectiveness has been evaluated in terms of BOD and suspended solids removals. The model utilizes the following values for combined sewage BOD and suspended solids as a function of time since the start of the storm event. These values are based upon data obtained during the study period.

Time since start of Storm	BOD mg/l	Suspended Solids mg/l
0 - 30 min.	143	348
30 - 60 min.	128	270
1 - 2 hours	97	193
over 2 hours	87	192

These parameters are not modified other than on the basis of the above time scale.

9. Removal Efficiency of Detention Tank - Performance efficiency of the tank is the result of both its ability to retain contaminants by virtue of the storage volume furnished, as well as separation by sedimentation during periods when overflow is taking place. The former factor is accounted for by the volumetric inventory maintained by the model. Removals by sedimentation are based on the following relationships developed from analysis of observed results.

% BOD Removal $25\ (1-e^{0.20t})$
% Suspended Solids Removal $= 40\ (1-e^{0.20t})$

Where t - time (hours) that the overflow is detained in the tank prior to overflow.

MODEL OUTPUT - ANALYSIS OF DETENTION TANK PERFORMANCE

A series of sixty (60) runs were made, using the detention tank model, with a year's hourly rain data as input. The effect of a range of variables was explored by modifying either the input rainfall data, or certain aspects of the storm water collection and detention tank system. The variables investigated by this program included the rain gauge used as the data source, the amount of precipitation per year, the emptying rate for the detention tank, tank size, interceptor size, and the relative effect of bypassing the tank when full. Table 39 summarizes the test conditions and model output for each of thirty sets of conditions. The model program was executed twice for each set to provide data on both BOD and suspended solids, including estimated removals.

Rain Gauge Data

As discussed in Section VIII, rainfall data was obtained from 3 independent stations during the program. Distinct differences were noted on some storm events and on the total amount of precipitation recorded. The significance of the difference in recorded rainfall on the tank performance is shown on Table 40. Station 1 (Airport U. S. Weather Bureau) compares favorably with station 2 (Holton Street Gauge in Project Area) in rainfall statistics over the period, and projected tank performance compares quite closely. Station 4 (Broadway Street Gauge Adjacent to the Project Area) rainfall data is substantially different from the other two stations and projections of tank performance are

TABLE 39

DETENTION TANK PERFORMANCE PROJECTIONS

Run	Period	Rain Sta.	Annual Rain (Inches)	Time to Empty Tank	No. of Events to Tank	Total Storm Hours	No. of Events Which Overflow Tank	Hrs. of Overflow	Hours of Rain	Vol. to Tank MG
1	11/1/71 thru 9/30/72	1	36.38 (11 mo.)	24 hr.	72	244	18	56	594	179
2	"	"	"	48 hr.	72	244	20	59	594	179
3	"	"	"	96 hr.	72	244	20	69	594	179
4	11/1/71 thru 10/31/72	2	35.53 (12 mo.)	24 hr.	74	261	18	58	488	181
5	"	"	"	48 hr.	74	261	19	61	488	181
6	"	"	"	96 hr.	74	261	20	63	488	181
7	"	3	28.88 (12 mo.)	24 hr.	[illegible]	224	13	48	464	144
8	"	"	"	48 hr.	[illegible]	224	15	55	464	144
9	"	"	"	96 hr.	62	224	17	59	464	144
10	'60 (Wet)	1	40.71	48 hr.	64	180	11	47	547	128
11	'70 (Normal)	"	28.85	48 hr.	70	226	10	43	584	124
12	'63 (Dry)	"	19.10	48 hr.	52	122	9	18	425	82
13	'60 (Wet)	1	40.71	48 hr.	64	180	33	[illegible]	547	128
14	"	"	"	48 hr.	64	180	24	73	547	128
15	"	"	"	48 hr.	64	180	9	31	547	128
16	'70 (Normal)	1	28.85	48 hr.	70	226	33	117	584	124
17	"	"	"	48 hr.	70	226	24	82	584	124
18	"	"	"	48 hr.	70	226	4	15	584	124
19	'63 (Dry)	1	19.10	48 hr.	52	122	28	57	425	82
20	"	"	"	48 hr.	52	122	15	34	425	82
21	"	"	"	48 hr.	52	122	2	5	425	82
22	'60 (Wet)	1	40.71	48 hr.	64	180	11	47	547	128
23	'70 (Normal)	"	28.85	48 hr.	70	226	10	43	584	124
24	'63 (Dry)	"	19.10	48 hr.	52	122	9	18	425	82
25	'60 (Wet)	1	40.71	48 hr.	123	450	16	84	547	173
26	'70 (Normal)	"	28.85	48 hr.	123	485	16	75	584	178
27	'63 (Dry)	"	19.10	48 hr.	105	356	10	29	425	115
28	'60 (Wet)	1	40.71	48 hr.	43	121	10	38	547	102
29	'70 (Normal)	"	28.85	48 hr.	52	142	7	29	584	93
30	'63 (Dry)	"	19.10	48 hr.	40	87	7	12	425	64

RUNS 22-23 UTILIZE BYPASS OPTION IN TANK PROGRAM
*RUNS 10-21 UTILIZE VARYING TANK SIZE AND PUMP OUT RATE IS VARIED TO MAINTAIN TIME TO EMPTY TANK.

TABLE 1[illegible]
(Cont.)

DETENTION TANK PERFORMANCE PROJECTIONS

Run	1000 lbs. BOD to Tank	Vol. Leaving Tank (MG)	1000 lbs. BOD Leaving Tk.	Percent Removal BOD	1000 lbs. S.S. to Tank	1000 lbs. S.S. From Tk.	Percent Removal S.S.	Percent Volume Retained	Tank Capacity (MG)	Interceptor Capacity
1	145	57	44	69.8	315	89	71.9	65.1	4.0	4xDWF
2	145	60	46	68.2	315	93	70.8	63.4	4.0	4xDWF
3	148	64	52	64.9	315	104	67.1	61.4	4.0	4xDWF
4	147	57	45	69.4	321	93	71.0	65.5	4.0	4xDWF
5	147	59	47	68.1	321	96	69.9	62.8	4.0	4xDWF
6	147	64	49	66.2	321	101	68.3	64.6	4.0	4xDWF
7	115	40	29	75.6	260	59	77.3	72.2	4.0	4xDWF
8	115	44	32	73.2	260	64	75.4	69.4	4.0	4xDWF
9	115	[illegible]	[illegible]	[illegible]	260	70	73.2	[illegible]	4.0	4xDWF
10	105	36	23	78.9	227	65	71.9	64.1	4.0	4xDWF
11	100	31	21	78.5	219	43	80.7	75.0	4.0	4xDWF
12	69	13	10	85.5	150	20	86.8	84.1	4.0	4xDWF
13	105	73	[illegible]	[illegible]	227	153	33.9	27.3	1.0	4xDWF
14	105	71	53	49.4	227	109	52.1	44.5	2.0	4xDWF
15	105	26	19	81.7	227	37	83.6	76.1	6.0	4xDWF
16	100	65	64	36.2	219	135	38.4	31.5	1.0	4xDWF
17	100	51	47	[illegible]	219	91	58.3	50.8	2.0	4xDWF
18	100	17	12	88.7	219	24	89.2	85.3	6.0	4xDWF
19	69	77	43	37.5	150	91	39.0	35.4	1.0	4xDWF
20	69	39	28	59.5	150	58	61.1	57.3	2.0	4xDWF
21	69	3	2	96.4	150	4	97.0	96.3	6.0	4xDWF
22	105	46	35	66.2	227	76	66.7	64.1	4.0	4xDWF
23	100	31	23	77.2	219	50	77.1	75.0	4.0	4xDWF
24	69	13	10	85.5	150	21	85.6	84.1	4.0	4xDWF
25	142	62	44	69.2	308	86	72.0	64.2	4.0	2xDWF
26	143	39	33	76.6	314	67	78.6	72.5	4.0	2xDWF
27	96	21	16	83.3	210	31	85.2	81.7	4.0	2xDWF
28	84	34	24	71.6	181	48	73.3	66.7	4.0	6xDWF
29	75	20	14	80.9	164	29	82.3	78.5	4.0	6xDWF
30	54	7	6	89.5	118	11	90.5	89.1	4.0	6xDWF

Table 40. SIGNIFICANCE OF SOURCE OF RAINFALL DATA

	Airport Gauge Station 1	Holton Gauge Station 2	Broadway Gauge Station 4
Period	11/01/71- 09/30/72	11/01/71- 09/30/72	11/01/71- 09/30/72
Inches of Rain	36.38	32.35	27.42
Number of Storms			
To Tank	72	67	53
To Stream	20	18	14
Hours with Flow			
To Tank	244	231	201
To Stream	59	49	49
Storm Water (mg)			
To Tank	179	168	134
To Stream	60	53	40
Percent Volume Retained	66.5	68.5	70.0
BOD Load (1,000 lb)			
To Tank	145	137	111
To Stream	46	43	30
Percent BOD Removal	68.2	68.9	73.5
SS Load (1,000 lb)			
To Tank	315	299	245
To Stream	93	89	60
Percent SS Removal	70.5	70.3	75.5

NOTE: Using 48 hour pump out rate and tank volume of 4.0 mg.
Int Cap = 4 x DWF.

significantly different. Since Station 2 and 4 are located quite close together relative to Station 1, the implications are (a) airport rain data, for which a substantial historical record exists can be used in evaluation of the performance of a detention tank in the City of Milwaukee with satisfactory accuracy, and (b) the discrepancy shown by Station 4 rain data is most likely due to the inherent inaccuracy of the recorded data and the variations in rainfall which occured between the various station locations.

During the period November 1971 thru 1972, in which the Humboldt Avenue detention tank was in service, it is estimated that a total of approximately 68.5 percent of the BOD and 70 percent of the suspended solids in inflows reaching the tank were intercepted and prevented from reaching the Milwaukee River, as indicated by this analysis. These estimates are based on an assumed pump out rate such that the tank would be emptied in 48 hours. In practice however, this rate was quite variable.

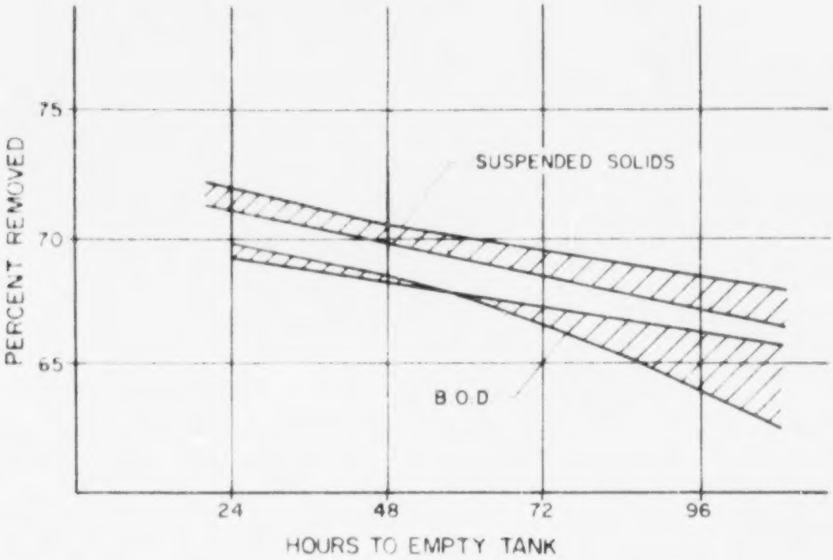
Effect of Pump Out Rate

The significance of pump out rate was examined by comparing projections of tank performance during 1972, with assumed pump out rates of 24, 48 or 96 hours. Results of these projections are illustrated by Figure 40. The comparison shown has been restricted to Station 1 and 2 because of the lack of correlation obtained at Station 4.

Pump out rate, expressed as hours of pumping required to empty the 3.9 million gallon detention tank is shown to have an effect on overall tank performance. Suspended solids removal was reduced by about 4 percent (67.5 percent removal versus 71.5 percent) as emptying time increased from 24 to 96 hours. BOD removals were affected by approximately the same magnitude (65 percent versus 69.5 percent).

While the pump out rate is an important factor, its effect within the range shown does not appear to be major when annual performance is evaluated. The use of a 96 hour period would increase the amount of BOD discharged to the river by about 5,000 pounds (45,000 versus 50,000 pounds/year) and suspended solids by about 11,000 pounds (91,000 versus 102,000 pounds/year). The longer emptying rate would therefore increase pollutant discharge by approximately 10 percent compared with what could be achieved with a 24 hour pump out time. On certain individual storms however the effect would be substantial.

FIGURE 40 - EFFECT OF PUMP-OUT RATE ON DETENTION TANK PERFORMANCE



Rain Station	Period	Pumpout Time (Hrs)	Percent of Applied Load Entering Riv.		Percent of Applied Load Removed	
			BOD	S.S	BOD	S.S.
1	Nov. 1-71 to Sept. 30-72	24	30.2	28.1	69.8	71.9
1	" "	48	31.8	29.5	68.2	70.5
1	" "	96	36.0	32.9	64.0	67.1
2	Nov. 1-71 to Oct. 31-72	24	30.6	29.0	69.4	71.0
2	" "	48	31.9	30.1	68.1	69.9
2	" "	96	33.8	31.7	66.2	68.8

Effect of Tank Volume

The effect of tank size on overall performance in reducing overflows on pollutant loads reaching the river has been explored in a series of runs which project performance for detention tanks ranging in size from 1 to 6 million gallons. The comparison has been made reducing tank volume to million gallons/square mile of drainage area by correcting for the test area (570 acres = 0.9 square miles). To compare relative efficiency with various amounts of precipitation, test runs were made using rain data from a wet, a normal, and a dry year. The years selected were as follows:

1960 - Wet Year - 40.7 inches
1970 - Normal Year - 28.9 inches
1963 - Dry Year - 19.1 inches

The results of this analysis are illustrated on Figures 41 (BOD), and 42 (suspended solids).

Detention tank size is seen to be of major significance. The larger the tank, the greater the removal, and over the range of sizes explored, no sharp break in performance versus size is noted. In the larger range of tank volumes, significant differences in effectiveness are noted between wet and dry years. This is not the case with smaller volume tanks where the amount of precipitation has a minor effect on efficiency.

BOD removal efficiencies of 30 to 35 percent are projected for tank volumes of about 1 mg/sq mi, which increases to a range of 80 to 95 percent for tank volumes of 6 mg/sq.mi. Comparable effects are indicated for suspended solids.

Effect of Interceptor Capacity and Rainfall

Runs 25-30 of Table 39 shows the effect of interceptor capacity on the combined sewer overflow using a 48 hour tank pump out time and a 4.0 mg tank capacity. This effect is demonstrated for a wet, normal, and dry year of rainfall data. The data used also shows the effect of rainfall in that the hours of rain and number of storms was actually larger for the normal year than for the wet year even though the total inches is less than the wet year. Analysis indicates that this is the result of the incidence of higher intensity storms during the wet year. Under the weather conditions used for this analysis as the interceptor capacity increases from 2 times the dry weather flow to six times dry weather flow, the volume of water reaching the tank is decreased and likewise the BOD and suspended solids load to the tank will be

FIGURE 41
EFFECT OF DETENTION
TANK VOLUME ON
B.O.D. REMOVAL

PERCENT APPLIED TO TANK
REMAINING IN DISCHARGE TO RIVER

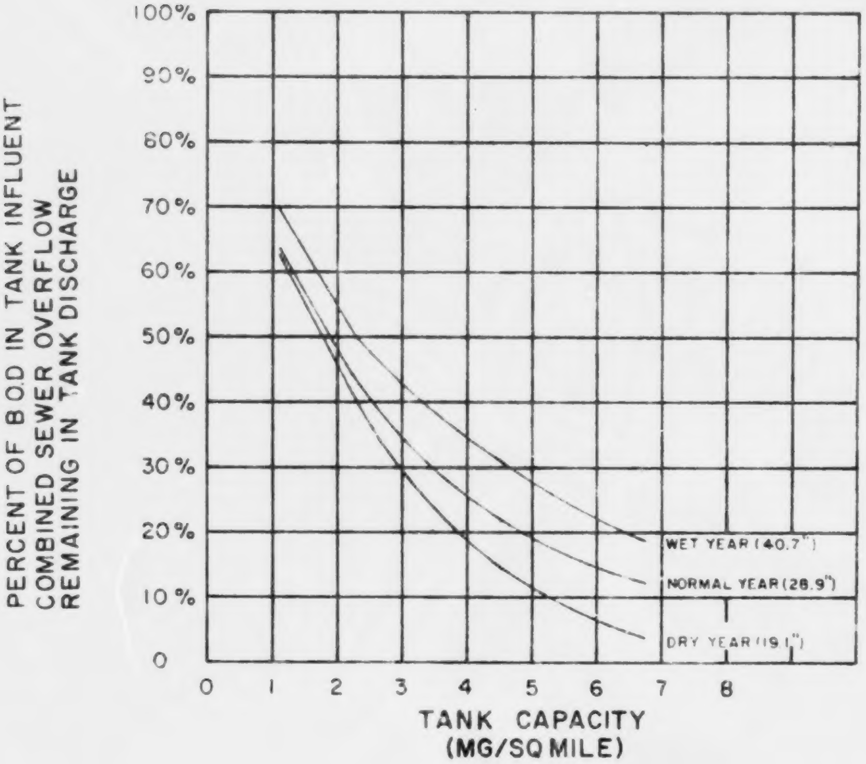
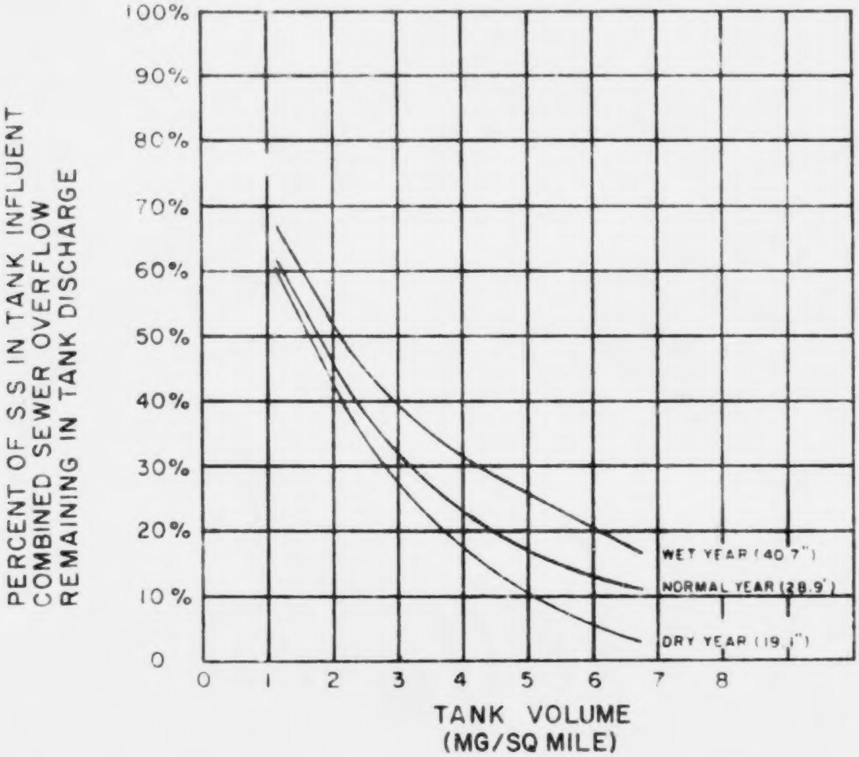


FIGURE 42
EFFECT OF DETENTION
TANK VOLUME ON
SUSPENDED SOLIDS REMOVAL

PERCENT APPLIED S.S. REMAINING
IN DISCHARGE TO RIVER



decreased (See Figure 45). If the load to the tank is lower the load to the stream will also be lower. Percent removals of BOD and suspended solids slightly increase as the interceptor capacity increases and loading to the tank decreases for the normal and dry year. During the wet year, the BOD and suspended solids loadings to the tank are almost the same as those of the normal year, but the loadings to the stream are about 10,000 lbs to 20,000 lbs higher during the wet year. The added loadings are, however, the result of less rainfall (total inches) falling over more hours for the normal year, which reduces the possibility of tank overflow an increases the percent volume of storm water retained. The percentage volume being 75 percent for the "normal" year as compared to 64.1 percent for the wet year as shown in run Nos. 10 and 11 in Table 39. The difference in percent BOD and suspended solids removal for the wet year for the different interceptor capacities is not as pronounced as the normal and dry year (See Table 41), due to the fact that 6xDWF is a relatively smaller percent increase over 2xDWF when compared to the total storm water volumes during a wet year than a dry year.

Effect of Bypass Operation of the Detention Tank

Studies were made investigating operation of the detention tank in a by-pass mode. That is, in a manner by which combined sewage influent to the tank is by-passed directly to the River, when the tank is full. This method of operation is compared to the normal operation, whereby all flow goes through the detention tank and when the tank is full combined sewage flows through the tank in a plug flow manner and overflows to the River. In this case the combined sewage is settled and chlorinated.

Due to the plug flow nature of the detention tank, during certain storm conditions, the tank effluent can be higher in concentration of BOD and suspended solids than the tank influent. This results from the fact that the higher concentration of pollutants in the initial (first flush) flow to the tank; when reduced by sedimentation, are still higher than the concentrations of the more diluted, combined sewage which later enters the tank.

Table 42 compares for a wet, normal and dry year the performance of the storm detention tank for bypass and plug flow operation. This shows that on a yearly basis the plug flow operation has a slightly higher percent BOD and suspended solids removal. The bypass can be more effective than plug flow for individual storm events but if operated over the entire year would be slightly less effective because it more often causes detrimental effects.

FIGURE 43
EFFECT OF INTERCEPTOR CAPACITY ON B.O.D. & S.S. LOADING TO DETENTION TANK

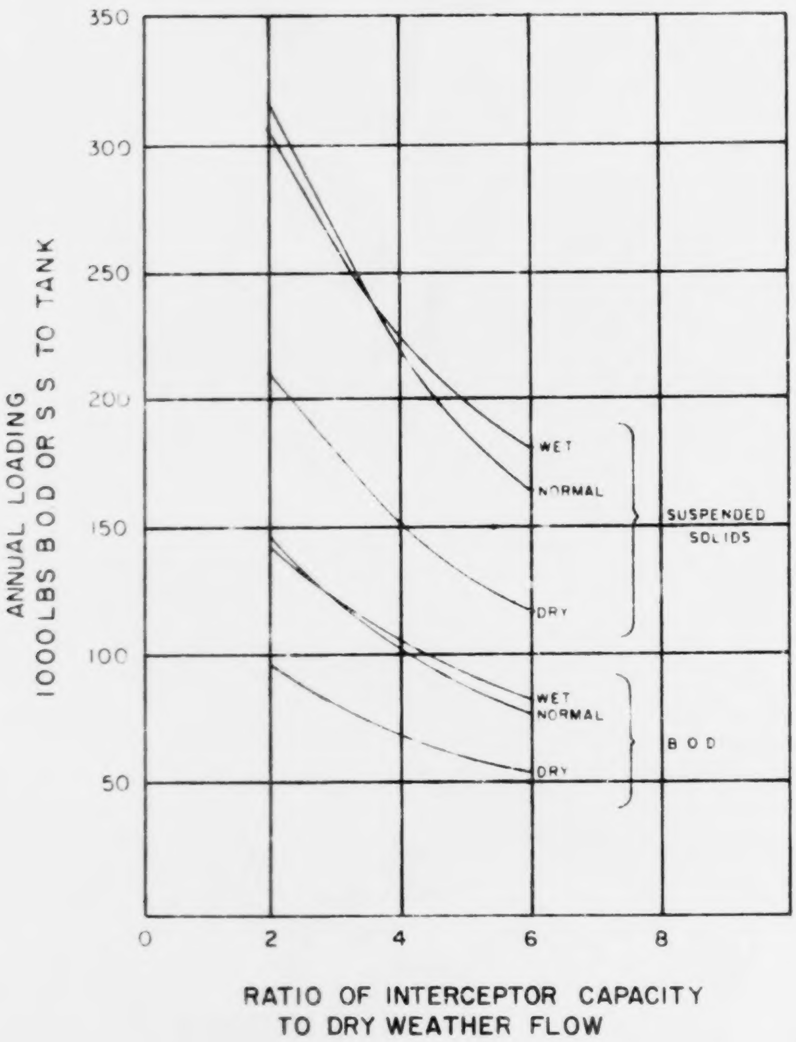


TABLE 41

EFFECT OF INTERCEPTOR CAPACITY ON COMBINED SEWER OVERFLOW (USING 48 HOUR PUMP OUT TIME AND TANK CAPACITY OF 3.9 MG)

		WET YEAR			NORMAL YEAR			DRY YEAR		
PERIOD		1960			1970			1963		
RAIN (INCHES)		40.7			28.9			19.1		
RAIN (HOURS)		547			584			425		
NO. OF STORMS		64			70			52		
INTERCEPTOR CAPACITY		2xDWF*	4xDWF	6xDWF	2xDWF	4xDWF	6xDWF	2xDWF	4xDWF	6xDWF
No. of Storms	To Tank	123	64	43	123	70	52	105	52	40
	To Stream	16	11	10	16	10	7	10	9	7
Hours with Flow	To Tank	450	186	121	485	226	142	356	122	87
	To Stream	84	47	38	75	43	29	29	18	12
Storm Water (MG)	To Tank	173	128	102	178	124	93	115	82	64
	To Stream	62	46	34	49	31	20	21	13	7
BOD (1000#) Load	To Tank	142	105	84	143	100	75	96	69	54
	To Stream	44	33	24	33	21	14	16	10	6
% BOD Removal		69.2	68.9	71.0	76.6	78.5	80.9	83.8	85.5	89.5
S.S. (1000#) Load	To Tank	308	227	181	314	219	164	210	150	118
	To Stream	86	65	48	67	43	29	31	20	11
% S.S. Removal		72.0	71.5	73.3	78.6	80.3	82.3	85.2	86.6	90.5

* DWF (DRY WEATHER FLOW) = 2 MGD

TABLE 42

COMPARISON OF BYPASS OPERATION OF DETENTION TANK VERSES PLUG FLOW OPERATION
(USING 48 HOUR PUMP OUT TIME AND 3.[illegible] MG TANK CAPACITY)

		WET YEAR		NORMAL YEAR		DRY YEAR	
PERIOD		1960		1970		1963	
		Plug Flow	Bypass	PF	Bypass	PE	Bypass
BOD	TO TANK	105	105	100	100	69	69
LOAD (1000#)	TO STREAM	33	35	21	23	10	10
% BOD REMOVAL		68.9	66.2	78.5	77.2	85.5	85.5
S.S.	TO TANK	227	227	219	219	150	150
LOAD (1000#)	TO STREAM	65	67	43	50	20	21
% S.S. REMOVAL		71.5	66.7	80.3	77.1	86.8	85.6

The bypass method of operation was investigated to evaluate the effect on removal efficiency of BOD and suspended solids. In practice the operation of the tank in this manner, even during certain individual storms may not be desirable because of the fact that a satisfactory chlorine contact time would not be available if the detention tank were bypassed. However, it would be desirable to study and evaluate the practicality of including this by-pass mode option in future detention tanks at other locations. Possibly an alternate chlorination scheme could be included in these future tanks which would allow utilizing the by-pass mode during certain selected storms and still allow for disinfection of the bypassed combined sewage.

RIVER WATER QUALITY MODEL

Intensive River Water Quality Data

Four intensive surveys of a duration of greater than five consecutive days were conducted during the following four periods:

Survey I	October 2-7, 1970
Survey II	September 11-16, 1970
Survey III	May 17-25, 1972
Survey IV	August 16-24, 1972

These surveys consisted of the bi-hourly measurement of dissolved oxygen and water and air temperature at the Humboldt Avenue Bridge (62), Cherry Street Bridge ([illegible]), St. Paul Avenue Bridge (52), Water Street Bridge ([illegible]), and the Flushing Tunnel intake (40). The dissolved oxygen content of the samples was determined by the Winkler titration method. All samples on the river were taken at a depth of approximately five feet below the water surface and the Flushing Tunnel intake samples were taken at a depth of approximately two feet below the water surface. During the latter two surveys, daily measurements of the water quality parameters previously mentioned in Section VIII at all water quality stations below the vicinity of North Avenue Dam were also incorporated into the sampling program.

Discharge data from two U. S. G. S. gaging stations were utilized to obtain river flow conditions during the above survey periods. Bi-hourly measurements were available at Station No. 4-0870 which is located on the Milwaukee River approximately 6.6 miles upstream from the mouth and gauging approximately 98% of the total drainage area. Daily discharge measurements were available at Station No. 4-0871.2 which is located on the Menomonee River approximately 6.2 miles upstream from the mouth. Flows in the Kinnickinnic River were estimated from a correlation based on limited data taken by U.S.G.S. during periods of base flow (little or no surface runoff).

"Dry" Weather Surveys

Surveys I and III are categorized as "dry" weather surveys. No rainfall or storm sewer overflows were recorded during these survey periods in the City of Milwaukee.

Survey I

During Survey I, bi-monthly flows ranging from 200 cfs to 138 cfs with an average of about 158 cfs were recorded at the Milwaukee River gaging station. Average Mean daily discharge in the Menomonee River was about 24 cfs with a range of 32 cfs to 20 cfs. The flushing tunnel was operated during the following periods:

Date	Time of Tunnel Operation
October 2	0800-1530
October 3	No flushing
October 4	No flushing
October 5, 6	0800-1530

The observed dissolved oxygen profiles for the four Milwaukee River station are presented in Figure 44 for Survey I. At Humboldt Avenue, the effect of tunnel operation can be readily seen by comparison of DO levels observed during the weekend with those observed during periods of tunnel operation. Super-saturated DO conditions observed at this station generally occured during tunnel operation. The utilization of DO is observed as one proceeds downstream. A minimum DO value of 2.65 mg/l was observed at Water Street.

Examination of the data shows no distinct pattern of diurnal DO variations. Percent possible sunshine for each of the survey days in chronological order were 80, 64, 65, 100, 42, based on climatological data taken at Mitchell Field. Based on these observations it appears that no significant algal activity occured in the Milwaukee River during the survey period, although there are times when large masses of algae cover the river surface.

Survey III

During Survey III, bi-hourly flows in the Milwaukee River decreased gradually from a high of about 360 cfs reached during the first survey day to about 230 cfs at the end of the survey. Average Milwaukee River

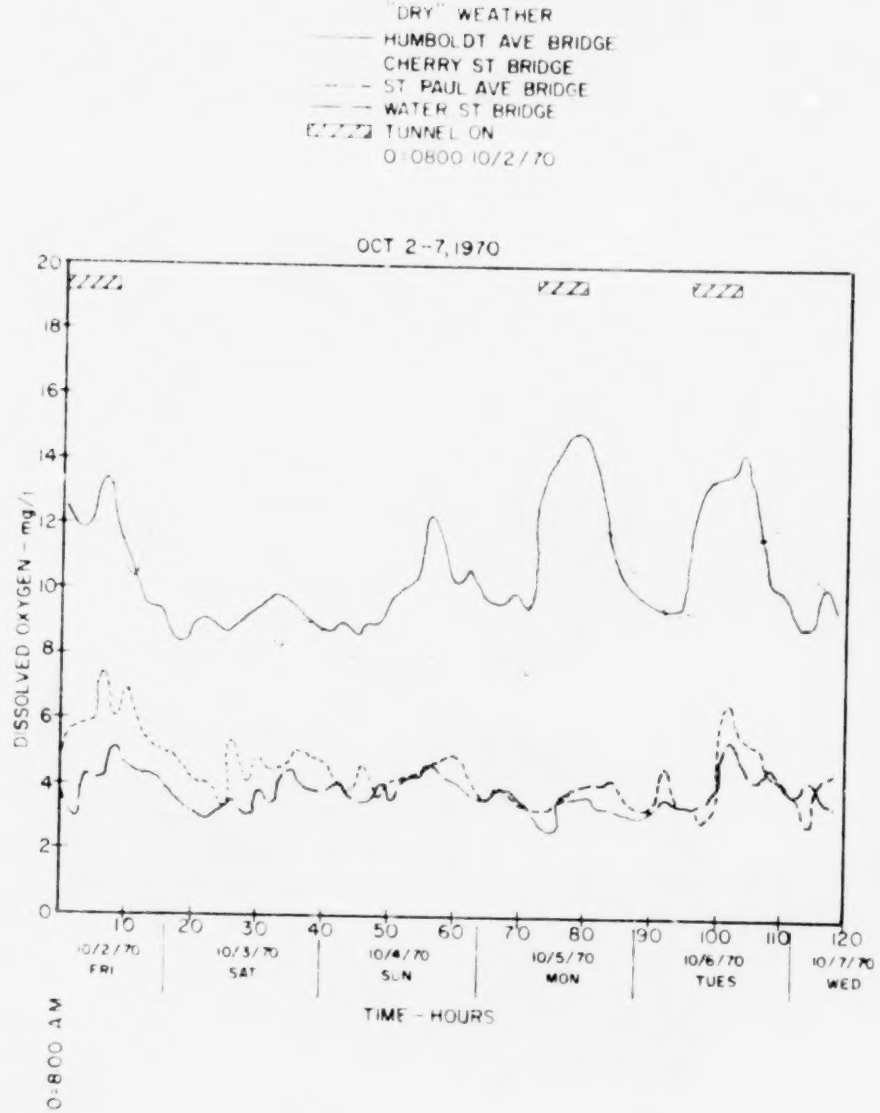


FIGURE 44
TEMPORAL D O DISTRIBUTION
SURVEY I

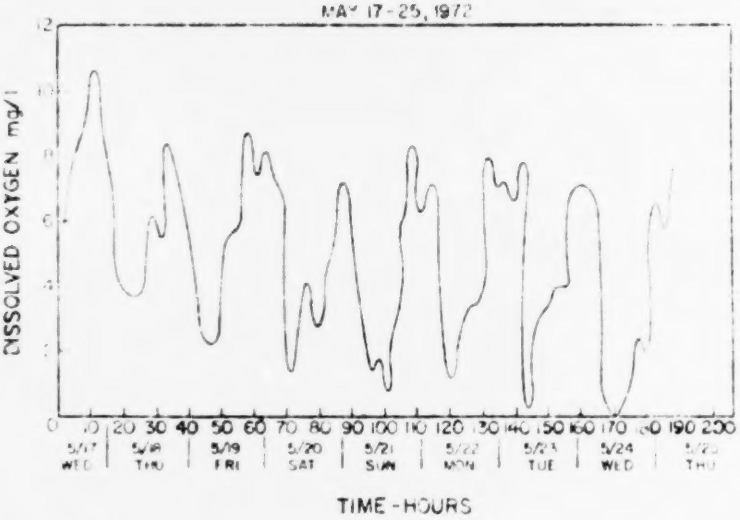
flow for the eight-day period was about 270 cfs. In the Menomonee River, average mean daily discharge was about 36 cfs with a range of 28 cfs, to 45 cfs. The flushing tunnel was not operated at all during the survey period. Percent possible sunshine for the survey days was greater than 93% except for May 20, 1972, when it was 75% as recorded at Mitchell Field.

In order to ascertain conditions at key locations in the study area, the sampling schedule was expanded over that of the previous dry weather survey. This expansion included intensive bi-hourly DO sampling at North Avenue Bridge (65) above the dam. River depth at the station limited sampling to a depth of approximately three (3) feet below the water surface. Three new stations in the Menomonee River (81), Kinnickinnic River (97) and the Milwaukee River Outlet to Harbor (47) were also incorporated into the daily sampling schedule. Daily measurements of all water quality parameters previously cited at all stations below North Avenue Bridge were also conducted.

The dissolved oxygen profile observed at the North Avenue station is presented in Figure 45. Figure 46 presents the DO profiles for the remaining Milwaukee River stations below the dam. Very wide variations in dissolved oxygen were observed at North Avenue Bridge which would usually be characteristic of high photosynthetic activity. However, the peak DO concentration generally occurred during no light periods between 8:00 p.m. and 6:00 a.m. At Humboldt Avenue, below the dam, these variations were attenuated, which can be attributed to the effect of the dam driving DO levels toward saturation plus DO added by the Flushing Tunnel. At Cherry Street, wide fluctuations in DO were again observed.

It can be postulated that the out-of-phase DO variations observed at North Avenue are a result of the transport of DO variation caused by photosynthetic activity occurring upstream. Under this hypothesis, the organisms producing this activity are assumed fixed or retained upstream (i.e., rooted aquatic plants). Reacration over the dam would attenuate DO fluctuations as observed at Humboldt Avenue. The reoccurrence of wide DO fluctuations at Cherry Street as specifically demonstrated late in the day on May 21 and 22 indicate the resurgence of a viable algal community. Yet during the first three days of the survey, peak DO levels occurred during night hours. Between Humboldt Avenue and Cherry Street, there are no recorded waste discharges. However, during the latter three days of the survey (Monday to Wednesday), a distinct deterioration of water quality as shown in Figure 46 occurred at Cherry Street. It is postulated that the inconsistent DO variations observed at Cherry Street are a result of a combination of factors, including the transport of DO variation

'DRY' WEATHER
NORTH AVE. BRIDGE (65)
0=0900 5/17/72



0=9:00 AM

FIGURE 45
TEMPORAL D.O. DISTRIBUTION
SURVEY III

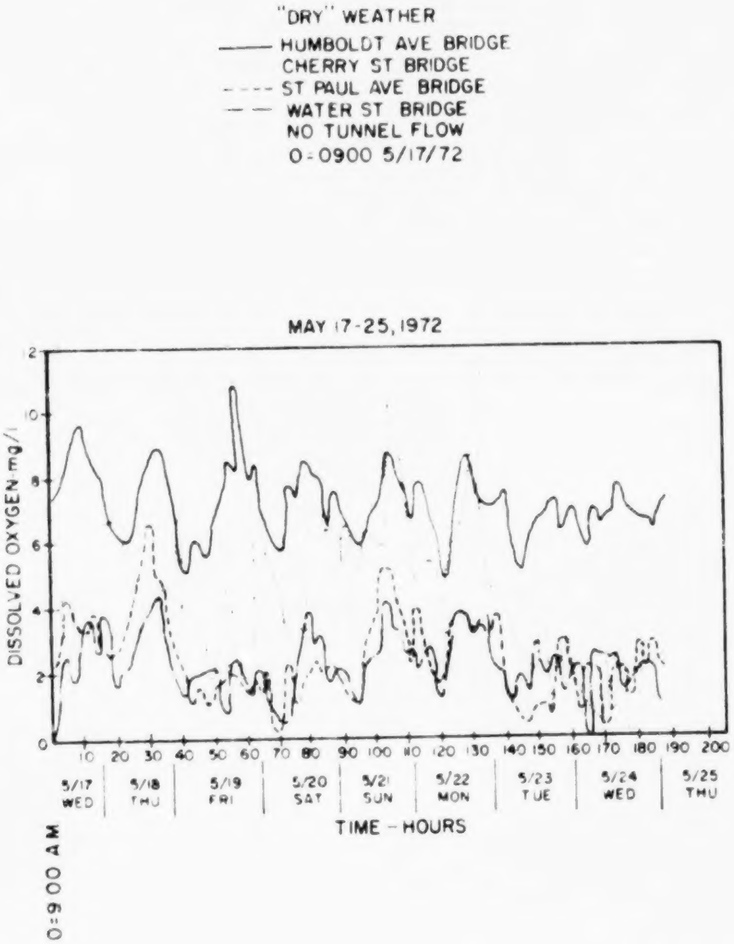


FIGURE 46
TEMPORAL D.O DISTRIBUTION
SURVEY III

as observed at Humboldt, photosynthetic activity which at times may have been depressed and some variable unknown source of oxygen utilization. It should be noted that at times the entire River area south of the Dam is an estuary of Lake Michigan and that solids can be trapped in the River area with oxygen required to stabilize the organic solids. The data available does not allow a differentiation of these parameters. Comparison of the DO profiles at St. Paul Avenue and Water Street with that at Humboldt Avenue demonstrate the overall degradation in water quality as measured by DO in the Milwaukee River. A minimum DO level of 0.0 was measured at Water Street during Survey III.

"Wet" Weather Surveys

Survey II:

Survey II, performed in September 1970, is categorized as a "wet" weather survey. A comparison of mean daily discharge in the Milwaukee and Menomonee Rivers and rainfall data taken at three key rain gaging stations, one in the study area, one adjacent to the project area and one at Mitchell Airport are presented in Table 43.

Table 43 shows that the majority of the rainfall occured on September 15. A maximum flow of 1530 cfs was recorded in the Milwaukee River on that date. During the survey period, the flushing tunnel was operated during the following periods:

Date	Time of Tunnel Operation
9/11/70	0800 - 1530
9/12/70	No Flushing
9/13/70	No Flushing
9/14/70	0800 - 1530
9/15/70	0800 - 1530

Percent possible sunshine for each of the survey days in chronological order were 100, 37, 0, 0, 4 and 78%, based on climatological data taken at Mitchell Field.

The observed dissolved oxygen profiles for the four Milwaukee River stations monitored intensely are presented in Figure 47. At Humboldt Avenue, a sharp increase in DO with peak values of 10.15 and 9.30 mg/l was observed during tunnel operation on September 11 and 14, respectively. The average DO during the weekend when the tunnel was not in operation was about 6.5 mg/l. From the observed data at Humboldt Avenue, it appears that tunnel operations had a comparatively diminished effect under the high river flows recorded

TABLE 43

COMPARISON OF RIVER DISCHARGE AND RAINFALL ACCUMULATION

(September 1970 Survey)
Survey II

Data	Mean Daily Discharge (cfs)		Accumulated Rainfall (in.)		
	Milwaukee Riv.	Menomonee Riv.	Station 2 (Holton Gauge)	Station 4 (Broadway Gauge)	Station 1 (Airport Gauge)
9/11/70	130	27	-	-	-
9/12/70	160	22	.24	.15	.16
9/13/70	167	64	.16	.16	.18
9/14/70	160	32	.24	.21	.22
9/15/70	547	628	1.26	.97	.86
9/16/70	239	76	-	-	-

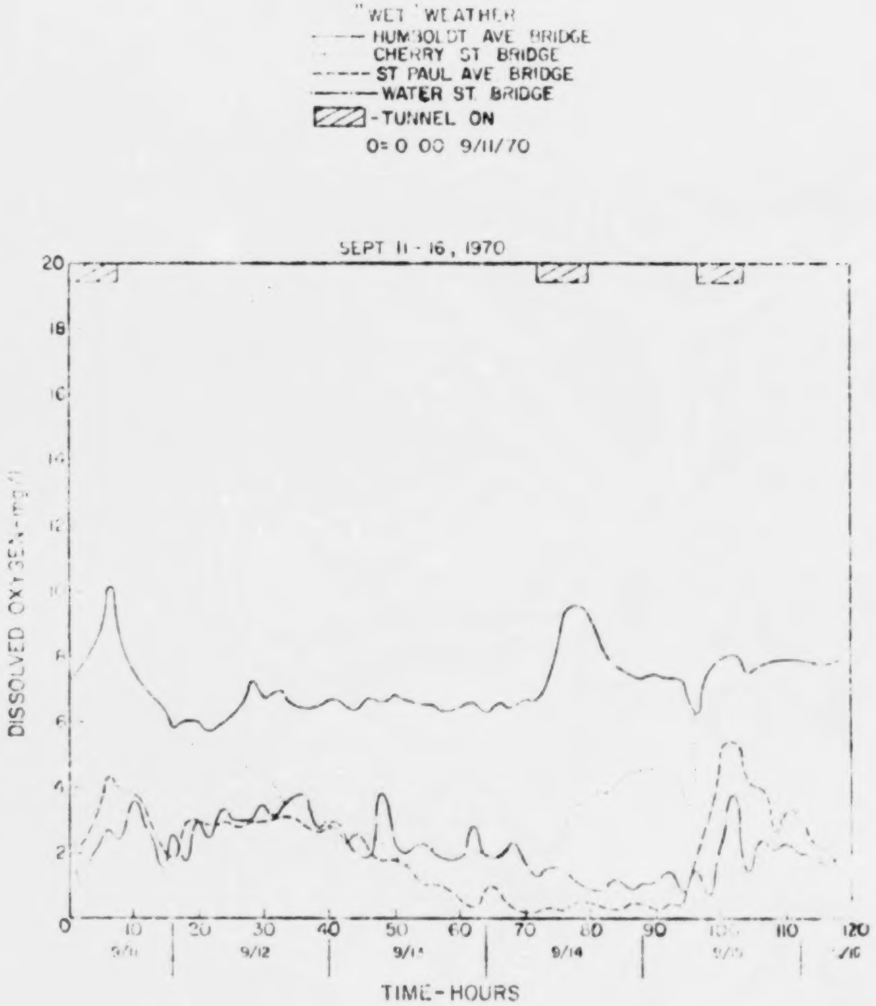


FIGURE 47
TEMPORAL D.O. DISTRIBUTION
SURVEY II

during September 15. The data at the three remaining stations show a temporal deterioration of DO levels with minimum values of 1.8, 0.2 and 0.9 at Cherry Street, St. Paul Avenue, and Water Street, respectively. It is noted that during the low DO periods, higher DO values were recorded at Water Street than at St. Paul Avenue. This can be either due to a higher DO level in the Menomonee or the propagation upstream of higher DO waters from the harbor. However no data were available for the Menomonee or the harbor locations during this survey period. During the initial hours of the large rainfall on September 15, DO levels at the last three stations increased.

It therefore appears that the initial effect of this rainfall was to increase DO levels. This may have been due to increased reaeration caused by the sharp increases in flow in combination with the flushing effect of this intense rainfall. The data also indicates no significant algal activity occurred during the survey period.

Survey IV:

A second wet weather survey (Survey IV) was performed in August 1972. River mean daily discharge and rainfall data taken during the Survey are presented in Table 44.

A maximum flow of 599 cfs was recorded in the Milwaukee River on August 23. The data demonstrates that flow in the Menomonee River can vary considerably more than that in the Milwaukee River under wet weather conditions. This was also observed in the previous wet weather survey (Refer to Table 43). During Survey IV, the flushing tunnel was operated during the following periods:

Date	Time of Tunnel Operation
August 16-19	0700 to 1500 and 1900 to 0300
August 20	0600 to 1400
August 21-25	0700 to 1500 and 1900 to 0300

With this frequent operation of the tunnel, the average flow in the Milwaukee River above the confluence with the Menomonee River during the survey amounted to about 714 cfs, exclusive of any runoff that occured below the Milwaukee River U.S.G.S. gaging station located in Esterbrook Park. The flow however may have been no greater than the amount noted because the sewers may no longer have been discharging into the River when the peak passed the U.S.G.S. gauge. This average flow is considerably higher than that for any of the previous surveys. It is noted that rainfall occured during only one of the first seven days of the survey. Therefore a major portion of the survey was conducted under a high flow-no-rain condition.

TABLE 44

COMPARISON OF RIVER DISCHARGE AND RAINFALL ACCUMULATION
(August 1972 Survey)
Survey IV

Date	Mean Daily Discharge (cfs)		Accumulated Rainfall (in.)		
	Milwaukee Riv.	Menomonee Riv.	Station 2 (Holton Gauge)	Station 4 (Broadway Gauge)	Station 1 (Airport Gauge)
8/16/72	575	170	-	-	-
8/17/72	535	122	-	-	-
8/18/72	432	98	-	-	-
8/19/72	439	167	0.28	0.34	0.35
8/20/72	338	100	-	-	-
8/21/72	351	80	-	-	-
8/22/72	358	99	-	-	-
8/23/72	364	64	0.48	0.40	0.43
8/24/72	491	442	-	-	-

Percent possible sunshine for each of the survey days in chronological order were 40, 83, 79, 80, 86, 85, 93, 10, and 76 based on climatological data taken at Mitchell Field.

Intensive DO sampling was conducted at four Milwaukee River stations and at the tunnel intake. Daily measurements of all water quality parameters previously cited at all stations below North Avenue Bridge were conducted. The observed DO profiles for the four Milwaukee River Stations monitored intensely are presented in Figure 48. Due to frequent utilization of the tunnel, its effect cannot be distinguished from the observed data at Humboldt Avenue. Water quality as measured by DO was generally good at Humboldt Avenue and Cherry Street with average DO values of 6.8 and 5.6, respectively. However, substantial deteriation of of dissolved oxygen was recorded at the two downstream stations. Average DO values for the survey period were 1.4 and 0.8 at St. Paul Avenue and Water Street respectively with anerobic conditions occurring at both stations for periods as long as twelve consecutive hours. These conditions were the worst recorded at these two stations in the monitoring program. It should be noted that anerobic conditions were observed before, during and after the rainfall period on August 18. It therefore appears that the zero DO conditions recorded are probably not soley due to storm water overflow. From the observed variations in the DO study it appears that there was some algal activity during this survey period.

RIVER WATER QUALITY MODEL DESCRIPTION

To provide a basis for the evaluation of the causitive elements of water quality in the Milwaukee River a two dimensional (lateral and longitudinal) model of the study area was developed. The basic principle upon which the model is based is the concept of continuity of mass; that is, the total mass of each material modeled must be accounted for, whether dispersed, transported with freshwater or net circulation, or reacted away if the material is nonconservative.

The materials balance can be described in mathematical terms by a differential equation of the following form:

$$\frac{\partial c}{\partial t} \quad E_x \frac{\partial^2 c}{\partial x^2} + E_y \frac{\partial^2 c}{\partial y^2} = U_x \frac{\partial c}{\partial x} - U_y \frac{\partial c}{\partial y} \pm S \qquad (1)$$

where:

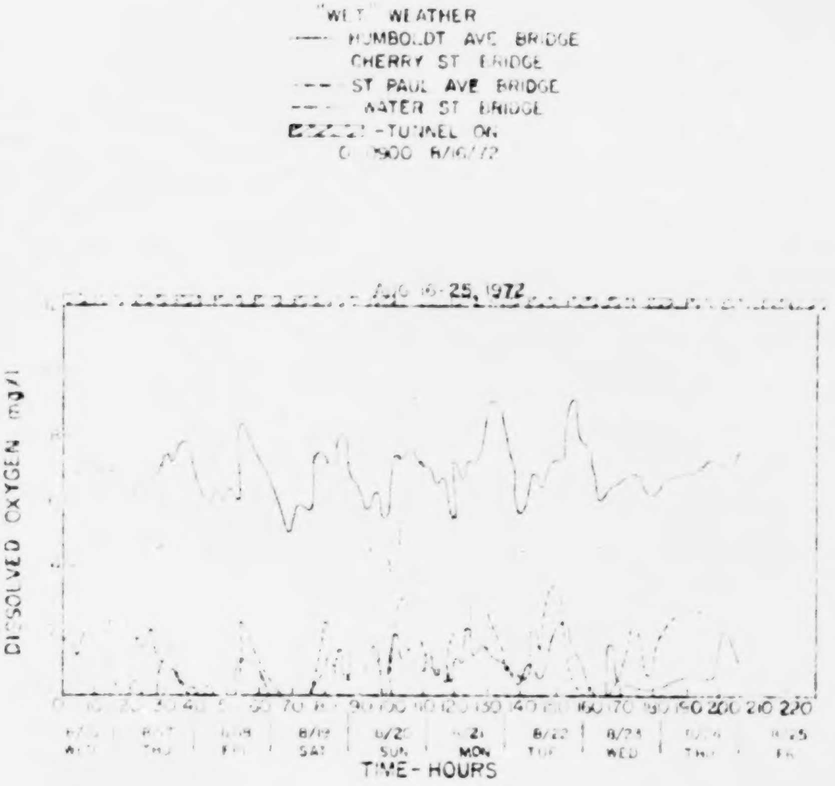


FIGURE 48
TEMPORAL D.O DISTRIBUTION
SURVEY IV

c	concentration of water quality
t	time
x	distance in longitudinal direction
y	distance in lateral direction
E_x, E_y	dispersion coefficients in the longintudinal and lateral directions, respectively
U_x, U_y	net advective velocity in longitudinal and lateral directions, respectively
S	all other sources and sinks of material, C

In large scale systems, it is convenient to apply mass balance, Equation (1), in a finite difference form. This procedure requires the division of the water body into a series of finite, interconnecting segments, and the application of a mass balance equation to each. The notation for the segments is shown in Figure 49. The mathematical model which results is a system of simultaneous linear ordinary differential equations of the form:

$$V_k \frac{dC_k}{dt} = \sum_j \left[-Q_{kj}(\alpha_{kj} C_k + \beta_{kj} C_j) + E'_{kj}(C_j - C_k) \right] -$$

$$V_k K_k C_k + W_k \qquad (2)$$

where:

C_k	concentration of water quality variable in segment k
V_k	volume of segment k
Q_{kj}	advective flow from segment k to segment j
α_{kj}	finite difference weight
β_{kj}	$1 - {}_{kj}$
E'_{kj} -	mixing coefficient between segments k and j $= \dfrac{E_{kj} A_{kj}}{L_{kj}}$
K_k	first-order reaction coefficients in segment k for water quality variable C
E_{kj}	dispersion coefficient between segments k and j
A_{kj}	cross-sectional area between segments k and j
L_{kj} -	average of characteristic lengths of segments k and j
W_{kj}	source (or sink) of variable C in segment k

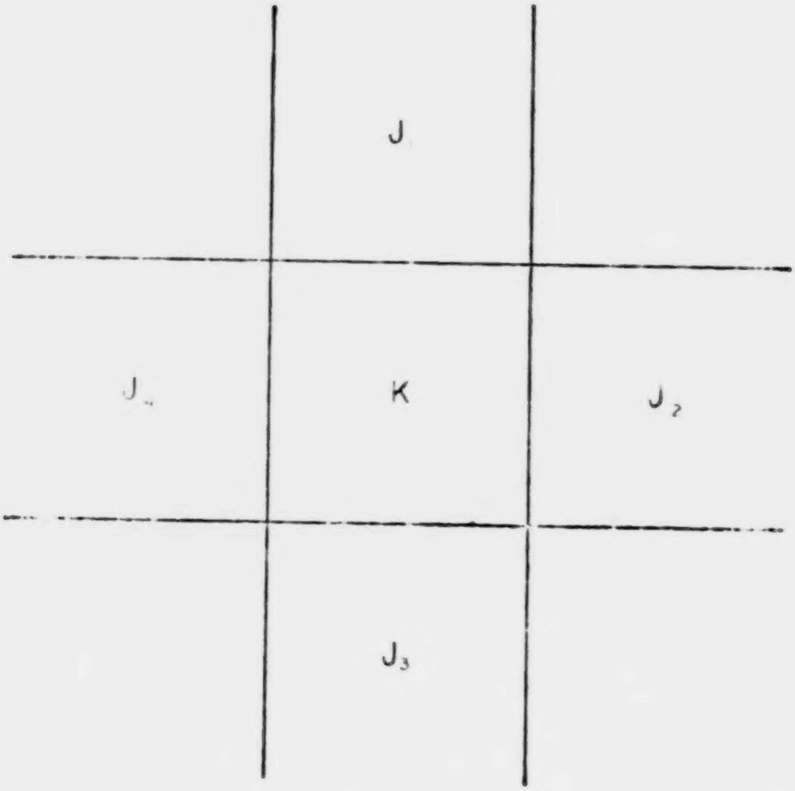


FIGURE 49
NOTATION FOR FINITE SEGMENTS

The finite difference form of Equation (2) is written for each segment. The equations provide for direct input of material into each segment as well as the biological utilization or production of material within a segment. Boundary values are established based on available data. The system is considered to be time variable, so that the finite difference equations are solved simultaneously at specified intervals of time using an Euler numerical integration scheme. The result of this integration is the concentration of mass calculated for each of the segments of the model at various times.

The left-hand side of Equation (2) represents the variation with time of the concentration C in segment k. The right-hand side of Equation (2) is made up of four parts: (1) the mass entering or leaving segment k as a result of the advective flow; (2) the mass dispersed into or out of segment k as a result of turbulent mixing; (3) the first order decay, if any, of the substance; and (4) the direct sources and sinks of the substance for segment k.

The area of study is presented in Figure 50. A grid of 111 segments was utilized and covered the Milwaukee River from the flushing tunnel outlet to the harbor, portions of the Menomonee and Kinnickinnic Rivers, and Milwaukee Harbor to the existing breakwater as shown in Figure 51. Primary emphasis was placed on the Milwaukee River where the four major intensively monitored stations were located. Segments in the Milwaukee River were generally 300 feet in length. Those segments of special interest are listed below:

Segment Number	Location
1	Humboldt Avenue Bridge
18	Cherry Street Bridge
35	St. Paul Avenue Bridge
40	Water Street Bridge

The variables that were analyzed, utilizing the mathematical model, were biochemical oxygen demand and dissolved oxygen.

The general parameters used in the model were obtained from measured data in the area or through verification procedures. Dispersion is a measure of turbulent mixing which in the area of concern would be caused by lake seiches, wind effects, the gological and geometric characteristics of the system and molecular diffusion. The dispersion phenomenon would have the net effect of driving the system to a uniform spatial concentration level with the attenuation of concentration gradients. A dispersion coefficient of 0.5 mi^2/day was utilized for Milwaukee Harbor and the Kinnickinnic, Menomonee and lower Milwaukee Rivers. Above the confluence of the Menomonee

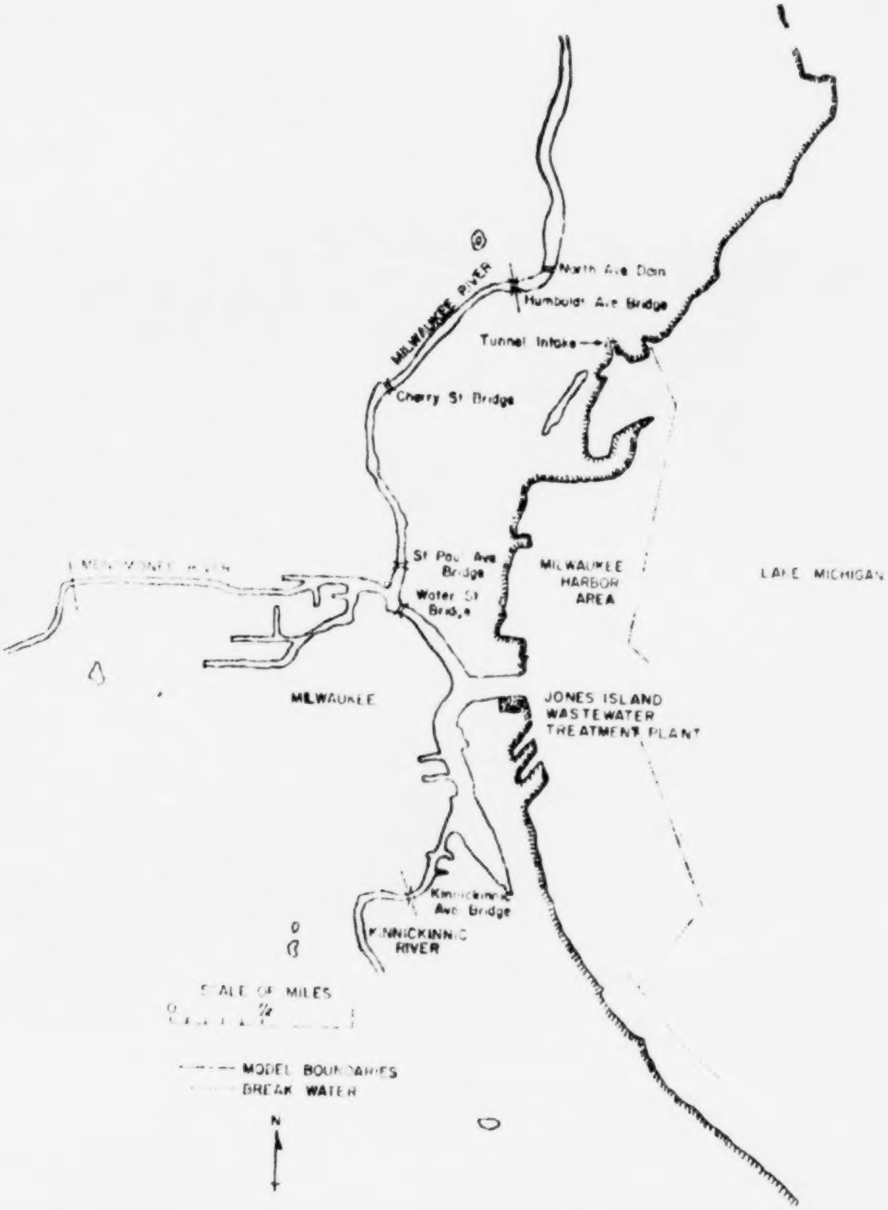

North Ave Dam
MILWAUKEE RIVER
Humboldt Ave Bridge
Tunnel Intake
Cherry St Bridge
St Paul Ave Bridge
Water St Bridge
MENOMONEE RIVER
MILWAUKEE HARBOR AREA
LAKE MICHIGAN
MILWAUKEE
JONES ISLAND WASTEWATER TREATMENT PLANT
Kinnickinnic Ave Bridge
KINNICKINNIC RIVER
SCALE OF MILES
0
½
1
MODEL BOUNDARIES
BREAK WATER
N

FIGURE 50
STUDY AREA

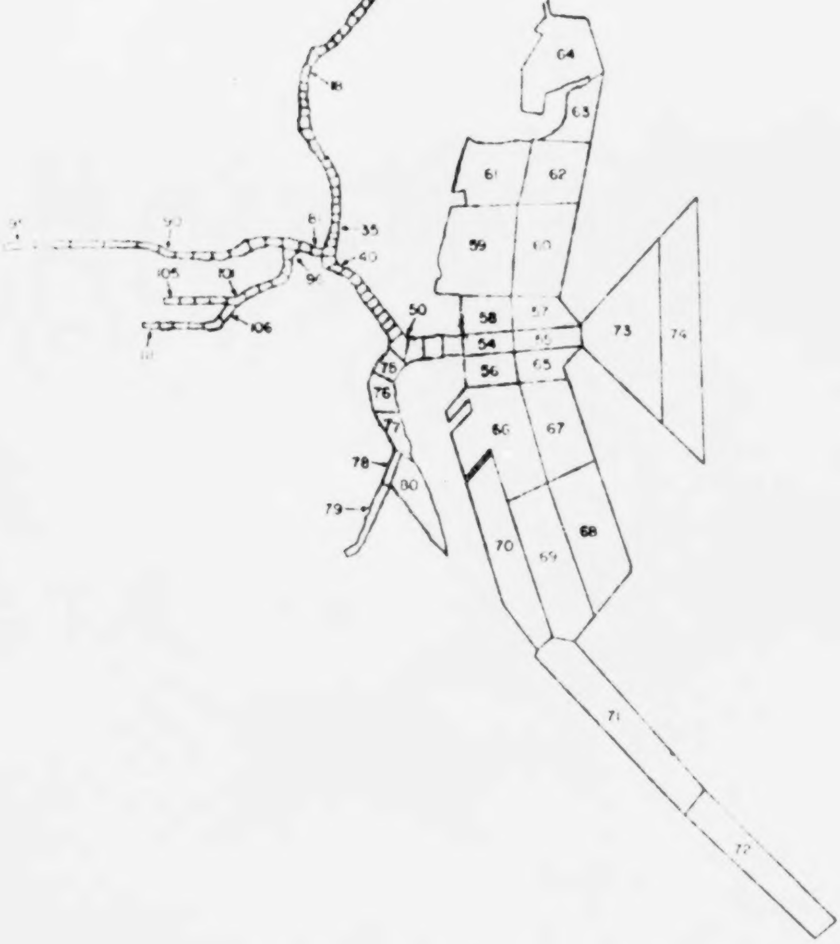

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FIGURE 51
MODEL SEGMENTATION

and the Milwaukee Rivers the dispersion coefficients linearly decreased from 0.- to 0.0 mi^2/day near the vicinity of the North Avenue Dam at Humboldt Avenue. For BOD, the deoxygenation coefficient (K_d), utilized in all model simulations was 0.20 at 20°C.

The reaeration coefficient in the rivers were approximated for each segment of the model from the following:

$$K_a = \frac{(D_L U)^{1/2}}{H^{3/2}}$$

in which D_L is the molecular diffusivity of oxygen in water (0.81 x 10^{-4}ft^2/hour at 20°C), U is the river velocity, and H is the mean stream depth. In the harbor segments, where velocities could not be well defined, a more basic formulation for the calculation of K_a was utilized:

$$K_a = K_L/H$$

in which K_L is the surface transfer coefficient (feet/day), which was assumed to be 2 feet/day in the model simulation. The aeration coefficient was not allowed to be less than 0.02/day under any flow regime. Both the deoxygenation coefficient and reaeration coefficients are functions of temperature and may be converted from their 20°C value in accordance with the following:

$$K_T = K_{20} (\urcorner)^{T - 20}$$

in which K_T is the rate value at any temperature, T, K_{20} is the 20°C value and $\urcorner$ is the empirical base of the relationship. For K_d and K_a, $\urcorner$ was assigned as 1.04 and 1.024 respectively.

A benthic oxygen uptake rate of 4 mg/m^2-day was measured in the laboratories of Marquette University for samples of Milwaukee River bottom sludges. This value was used in all river segments of the model. A benthal uptake rate of 1 gm/m^2-day was assigned to the harbor segments.

When photosynthetic oxygen production was included in the modeling analysis, this phenomenon was expressed as follows assuming a sinusoidal variation of oxygen production during the daylight hours:

$$P = P_m \sin\left(\frac{\pi t}{f}\right) \quad \text{for } 0 < t < f$$

$$P = 0 \quad \text{for } f < t < 24 \text{ hr}$$

in which P is the photosynthetic oxygen production rate at time, t, P_m is the maximum daily photosynthetic oxygen production rate, and f is the period of sunlight (hours). Respiration of aquatic plants, R, was incorporated into the model as a constant. Therefore, the net effect of aquatic plant life of the dissolved oxygen of the system is expressed as follows:

$$P_n = P - R$$

where P_n is the net production of oxygen by the aquatic plant community.

The geometry of the study area was based on U.S. Army Corps of Engineers Map No. 743, Milwaukee Harbor. Advective flows in the river was estimated from data taken by the U.S.G.S. at the river stations previously cited. The flushing tunnel flow rate was estimated to be 422 cfs based on dye studies.

The only known major waste discharge with the exception of combined sewer overflows and sewer by-passes in the study area is the Jones Island wastewater treatment plant. This plant provides secondary treatment and discharges the treated effluent to Milwaukee Harbor just beyond the mouth of the Milwaukee River. Average plant flow is approximately 160 MGD with an average BOD_5 load of approximately 20,000 pounds per day. Other wastewater treatment plants, bypasses, overflows, and cross connections, both known and unknown, exist in the Milwaukee River Drainage Basin and effect the study area water quality.

RIVER - WATER QUALITY MODEL OUTPUT AND VERIFICATION

The verification of Survey I (October 2 - 7, 1970) is demonstrated in Figure 52, 53, 54 and 55 where the observed and calculated dissolved oxygen profiles in the Milwaukee River are presented. The magnitudes of the model parameters such as dispersion, kinetic reaction rates, and benthal oxygen demand have already been presented in the previous section. The flow regime utilized was based on U. S. G. S. data combined with flushing tunnel operation.

Under dry weather conditions, the major factors contributing to the utilization of dissolved oxygen in the Milwaukee River have been estimated to be the benthal oxygen demand and the BOD_5 and DO deficit loads transported through the boundaries of the model. At Humboldt Avenue Bridge, the DO deficit boundary conditions were based on the actual bi-hourly data. Water quality measurements other than DO and temperature at the four primary Milwaukee River Stations were not taken during Survey I. The other boundary conditions of the model were estimated in this case. This was accomplished, in part, by evaluating all water quality data taken during the entire project. A BOD_5 boundary condition value of 5 mg/l was assumed at Humboldt

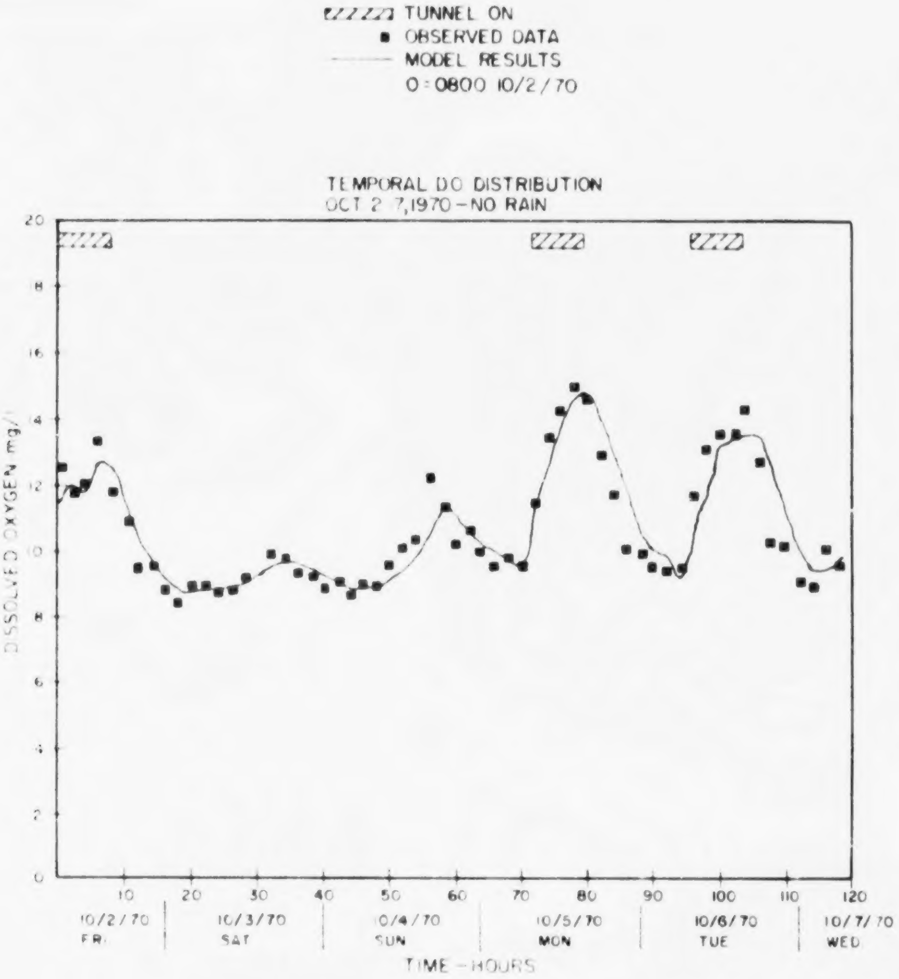


FIGURE 52
OBSERVED V.S. CALCULATED DATA
SURVEY I - HUMBOLDT AVE.
STATION 62

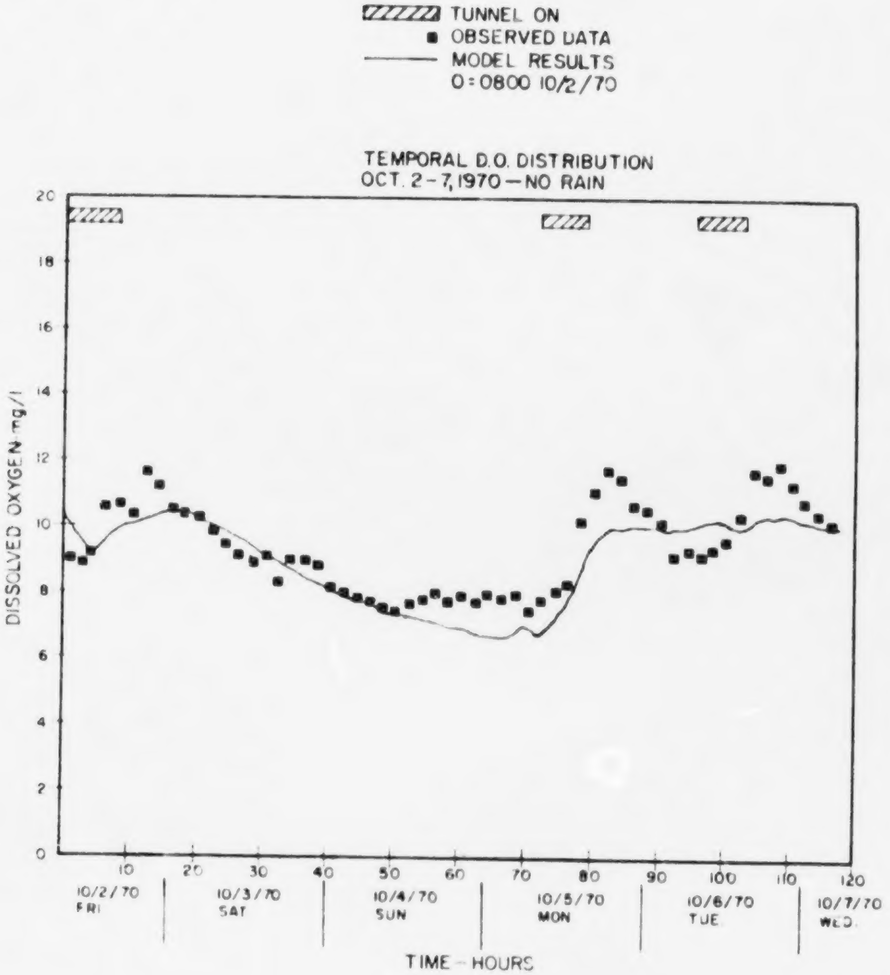


FIGURE 53
OBSERVED VS CALCULATED DATA
SURVEY I — CHERRY ST.
STATION 58

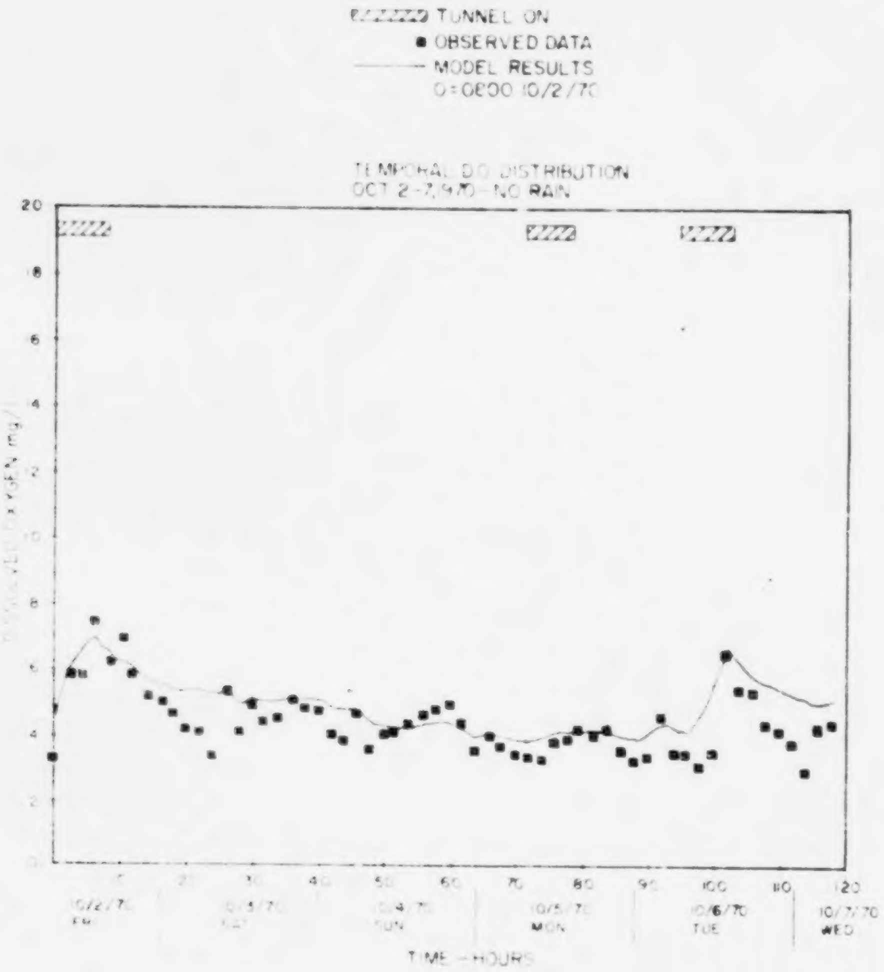


FIGURE 54

OBSERVED VS. CALCULATED DATA
SURVEY I – ST PAUL AVE
STATION 52

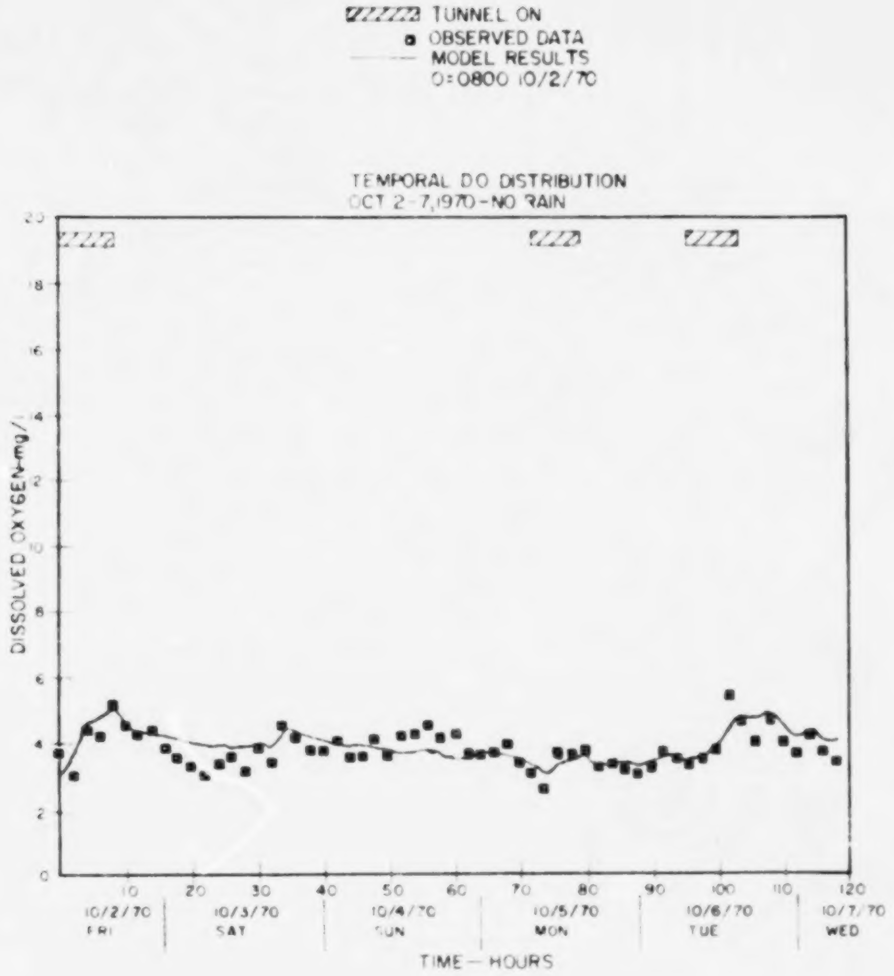


FIGURE 55
OBSERVED VS. CALCULATED DATA
SURVEY I-WATER ST
STATION 59

Avenue during the tunnel off periods. Surveys subsequent to Survey I indicated that the flushing tunnel water had a BOD_5 equal to approximately 80 percent of that measured at Humboldt Avenue and therefore, the appropriate dilute BOD_5 values were utilized during periods of tunnel operation. The values used at the other boundary locations are as follows:

Location	BOD_5 (mg/l)	DO Deficit (mg/l)
Menomonee River	2.5	2.0
Kinnickinnic River (Kinnickinnic Avenue Bridge)	5.0	10.0
Lake Michigan (At breakwater)	2.5	0.0

The higher DO deficit value estimated for the Kinnickinnic River boundary is based on data taken during later surveys which showed that oxygen deficit water existed at this location. The boundary conditions at the Menomonee River and Lake Michigan are respresentative of background levels.

The model verification for Survey I is considered to be very good. The boundary conditions at Humboldt Avenue accounted for approximately 59, 33, and 26 percent of the calculated DO deficits at Cherry Street, St. Paul Avenue, and Water Street respectively. It was estimated that approximately 36 percent of the calculated DO deficits at these three stations were due to the estimated bottom demand of 4 gm/m^2/day in the river sections. The model also indicated that the Jones Island Wastewater Treatment Plant had only a minor influence on DO levels at the monitoring locations under the conditions of Survey I. At Water Street the plant load contributed less than 6 percent of the total calculated DO deficit while having almost no effect at Cherry Street. The boundary conditions at Lake Michigan, the Menomonee River, and the Kinnickinnic River had a combined effect of contributing about 24 and 41 percent to the calculated DO deficit at St. Paul Avenue, and Water Street, respectively, while having a minimal effect at Cherry Street.

For Survey III, an expanded sampling schedule was initiated. As a result, a better definition of boundary conditions based on actual data taken during the survey was possible. Daily BOD_5 values at Humboldt Avenue varied from 6.2 to 9.6 mg/l with an average value of 7.9 mg/l which was utilized in the model. DO deficit boundary conditions at this station were based on bi-hourly data. At the Kinnickinnic River Station, daily measurements showed variations of 4.0 to 8.2 mg/l (average 5.3 mg/l) for BOD_5 and 6.5 to 9.9 mg/l (average 8.7) for DO deficit. These data were used to define the Kinnickinnic River boundary conditions. For the model simulation of Survey III, the same dispersion field, kinetic reaction rates, benthal oxygen demands, boundary conditions at the Menomonee River and Lake Michigan and average waste load from the Jones Island Wastewater Treatment Plant as that used in the Survey I simulation were applied. Background flows in the Milwaukee River were approximately 110 cfs greater than the flows recorded during Survey I. The flushing tunnel was not operated at all during Survey III. A comparison of observed and calculated data is presented in Figure 56 and 57 for the Milwaukee River.

Inputs into the model for simulation of Survey III were relatively constant. Flow in the Milwaukee River gradually increased from a high of [illegible] cfs during the first survey day to about 230 cfs at the end of the survey with no variation due to tunnel operation. The only significant variable input into the model was the DO variation at Humboldt Avenue. Since Survey III was conducted under dry weather conditions, no direct point waste sources other than the Jones Island Wastewater Treatment Plant were defined in the study area. As a result of the relatively constant nature of the inputs into the model, the DO variation input at Humbolt Avenue is attenuated as one proceeds downstream due to the effects of dispersion caused by chemical and physical reactions in the River, and the calculated profile becomes fairly flat at St. Paul Avenue and Water Street. The Model reproduces the general downward trend of the data quite well but is unable to follow the wide short-term fluctuations demonstrated by the data.

The observed data taken during Survey III demonstrated some peculiar features. Evidence of variable algal activity is indicated. Unfortunately, due to the original objectives of this project, the measurement of algae was not included in the sampling program and therefore, the simulation of the effect of algae can be no more than speculative. Nevertheless, in order to demonstrate the possible fluctuations in dissolved oxygen which could be caused by algae, a model simulation run of Survey III was repeated which included an expression of photosynthetic effects. A maximum daily photosynthetic rate of 20 mg/l-day was assumed to vary sinusoidally over a daily sunlight period of about 14 hours (based on climatological data taken at Mitchell Field). This model simulation also employed average respiration rates of 7.4 mg/l-day in all model segments. All other model inputs were the same as the previous simulation. The resulting calculated DO profiles are compared to the observed data in Figures 58 and 59. The observed DO variations are roughly approximated at St. Paul Avenue and Water Street. A similarity in the pattern of

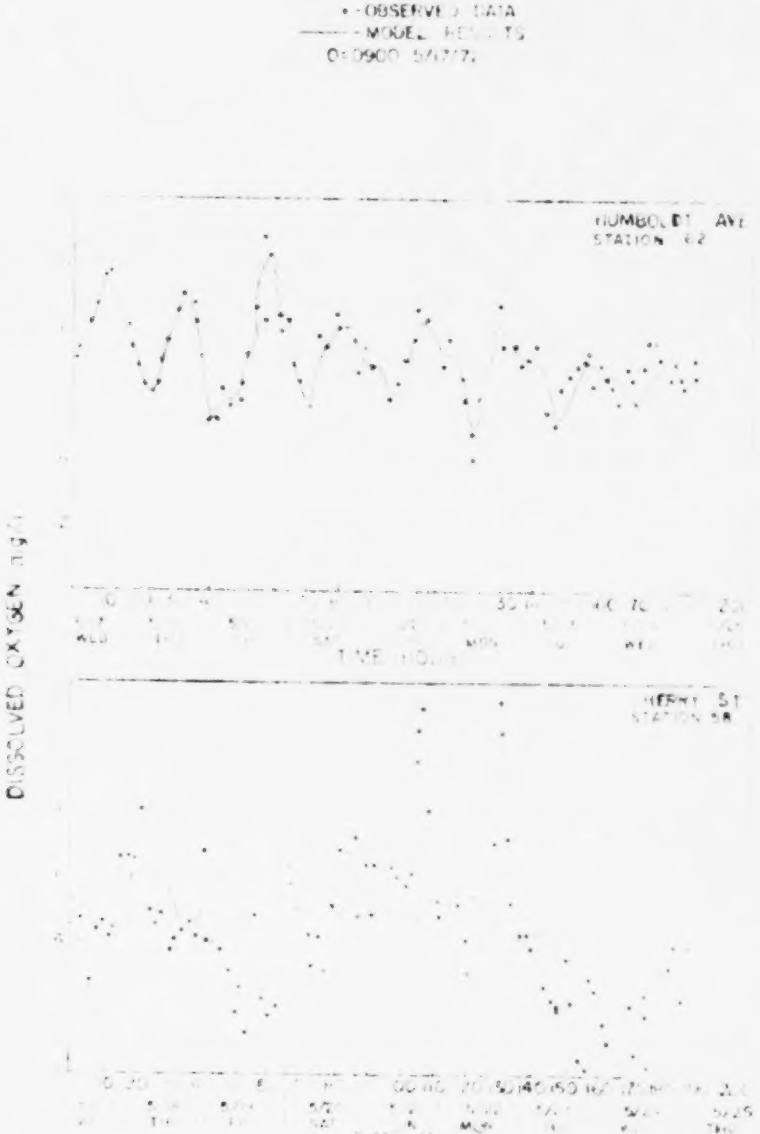


FIGURE 56
OBSERVED VS CALCULATED DATA
SURVEY III - HUMBOLDT AVE., CHERRY ST

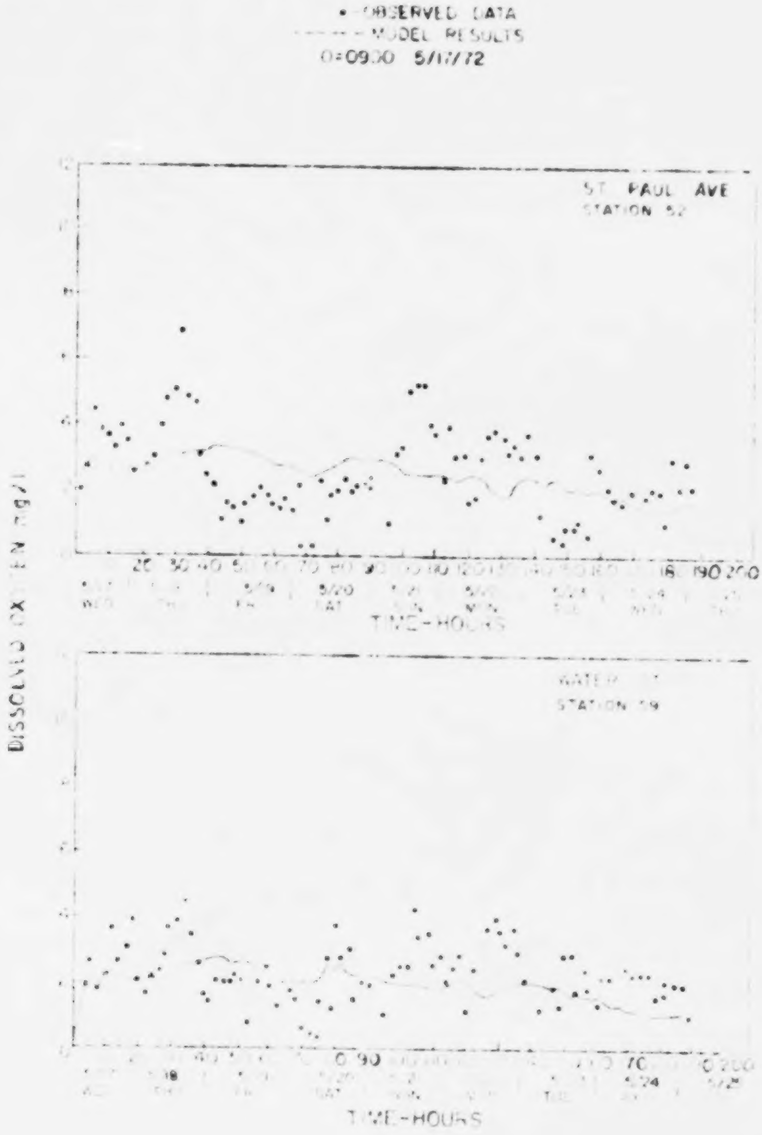


FIGURE 57
OBSERVED VS. CALCULATED DATA
SURVEY III ST PAUL AVE, WATER ST

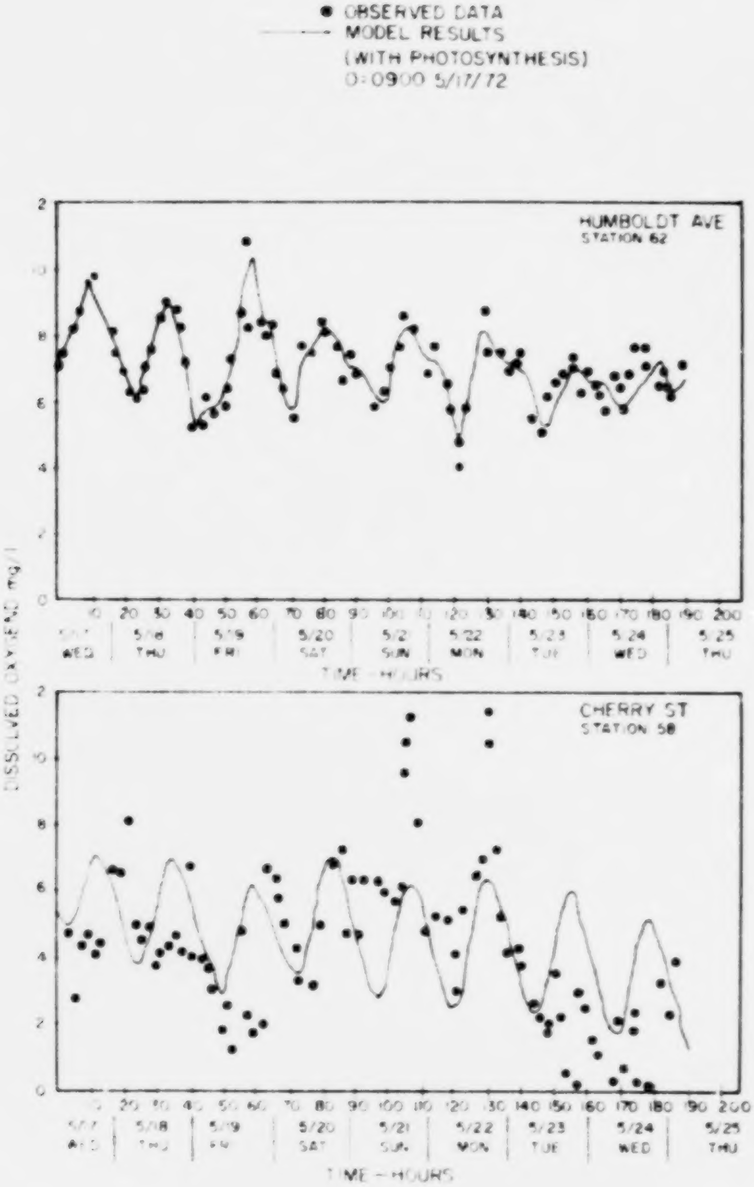


FIGURE 58
OBSERVED VS CALCULATED DATA
SURVEY III - HUMBOLDT AVE., CHERRY ST.

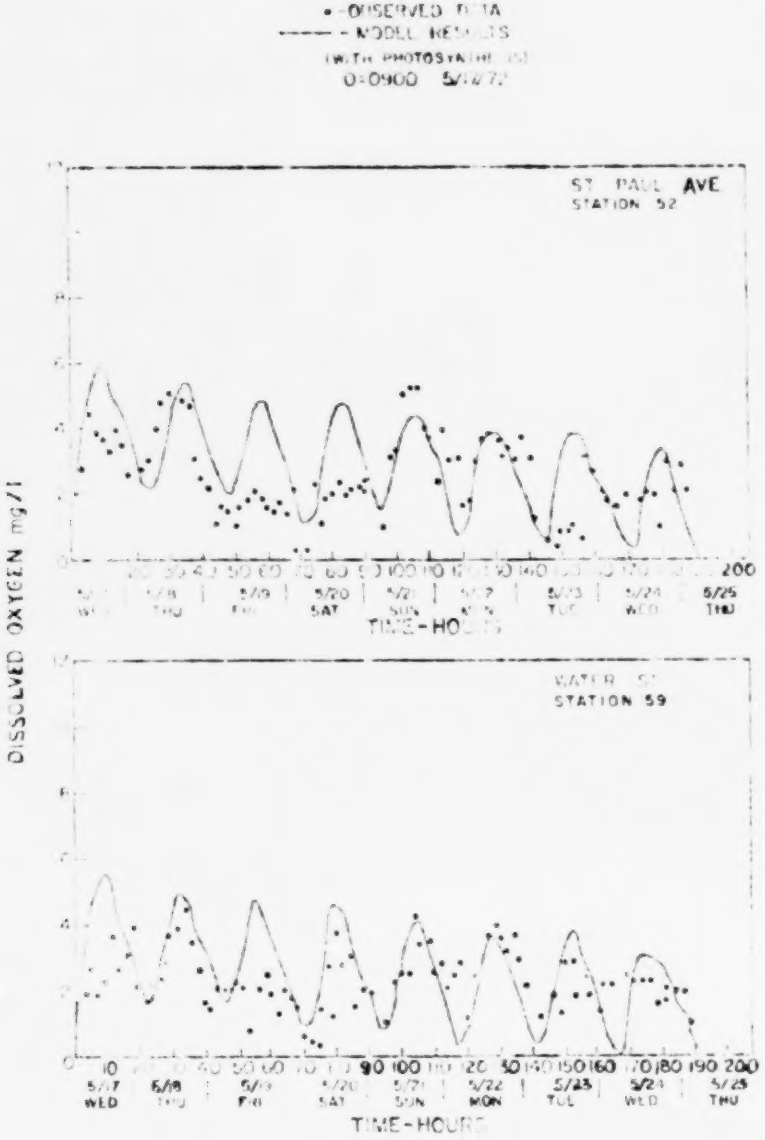


FIGURE 59
OBSERVED VS CALCULATED DATA
SURVEY III-ST PAUL AVE., WATER ST

fluctuation was also noted at Cherry Street although here the observed fluctuations are substantially greater than calculated by the model. It does appear from this exploratory analysis that short term fluctuations in DO observed during this survey, are in a significant measure due to algal activity in the Milwaukee River.

However, even without the assumed photosynthetic activity, the model does simulate the average of the observed data. Good agreement between observed and calculated oxygen levels has been obtained for two dry weather surveys using a consistent set of dispersion, kinetic, and benthal uptake parameters. The model has therefore been approximately verified under different dry weather flow regimes and thus provides an analytical framework for the preliminary deterministic evaluation of water quality in the Milwaukee River.

RIVER WATER QUALITY MODEL OUTPUT AND VERIFICATION

Wet Weather

Survey IV (August, 1972) was conducted under the expanded sampling schedule and as a result the major model boundaries could be defined based on actual data taken during the survey. During Survey IV rainfall was recorded on two of the nine survey days. Approximately 0.75 inches of rain fell during the survey. Daily BOD values at Humboldt Avenue varied from 4.2 to 8.4 mg/l for the first 8 days of the survey. During the last day of the survey a BOD of 15.8 mg/l was recorded. DO deficit boundary conditions at this station were based on bi-hourly data. BOD boundary conditions at the Kinnickinnic River were based on daily measurements which showed variations of 4.0 to 8.2 mg/l (average equals 5.0 mg/l). All DO measurements at the Kinnickinnic River station were 0.0 mg/l with a corresponding average DO deficit of approximately 8.6 mg/l.

Survey IV was first simulated without the inclusion of combined sewer overflows. The same dispersion field, kinetic reaction rates and benthal oxygen demands, developed and used in the dry weather simulation, were applied. Boundary conditions at the Menomonee River and Lake Michigan as well as the approximate loads from the Jones Island Wastewater Treatment Plant were also the same as those previously applied in the dry weather simulations. The flow rate regime utilized was based on U.S.G.S. data combined with flushing tunnel operation. No flows due to urban runoff from the immediate drainage area were included in the river flow and loads. The resulting DO profiles are compared with observed data in Figures 61 and 62 for the four key Milwaukee River stations. It can be noted that the observed data is fairly well simulated, although somewhat out of phase at Cherry Street. However substantial deviation between observed and calculated results is noted at St. Paul Avenue and Water Street.

A total of [illegible] combined overflows with an approximate drainage area of [illegible] acres drain into the Milwaukee River. Combined sewers below the North Avenue Dam serve approximately 2,600 acres of the total drainage area. During Survey IV, as previously stated, rainfall was recorded for 2 of the 9 survey days with a maximum sustained duration of 4 consecutive hours for any one storm. Approximately [illegible] inches of rain fell during the survey. In order to evaluate the effect of urban storm runoff on water quality in the Milwaukee River, it was necessary to determine the overflows and corresponding loads into the Milwaukee River for the recorded storms. The detention tank model was used in this regard to interface with the water quality model. The influent loads and flows to the tank from the 570 acre demonstration area were calculated for individual storms using the combined sewer overflow detention tank model. These estimates were then scaled to each combined sewer based on drainage area and inputed into the water quality model. For purposes of demonstrating the effect of storm conditions loads from the Jones Island sewage Treatment Plant were arbitrarily approximated to increase approximately 10 fold over average levels during periods of runoff. This loading was not based upon detailed data from the Treament Plant but was established arbitrarily.

The effects of combined sewer overflow due to the rainfall recorded on August 19 on DO levels in the Milwaukee River are shown by the dashed lines on Figures 60 and 61. As can be seen the effect of combined sewer overflow on DO is minimal under the conditions simulated. This may be due to the relatively short duration of the storms occurring during Survey IV combined with the reduced residence time (approximately one day) in the test area due to the frequent tunnel operation. Therefore the discrepancy between observed and calculated DO results at St. Paul Avenue and Water Street are not accounted for by combined sewer overflow occurring during the Survey period. The observed data also shows that anaerobic conditions occurred at these two stations for extended periods prior to the storm of August 19 as well as after. On the basis of the above it is concluded that other factors are controlling quality downstream. This conclusion is further reinforced by a comparison of the averages of data observed during Survey III (Dry Weather) with that of Survey IV (Wet Weather) as presented in Table 45.

The averages of BOD measured at Humboldt Avenue during both surveys were of a comparable order. Due to higher flows occurring during Survey IV as a result of frequent tunnel operation, reaeration in the Milwaukee River was calculated to be more than twice that calculated for Survey III. Under similar conditions (i.e., boundary conditions at Humboldt Avenue) it would be expected that lower DO

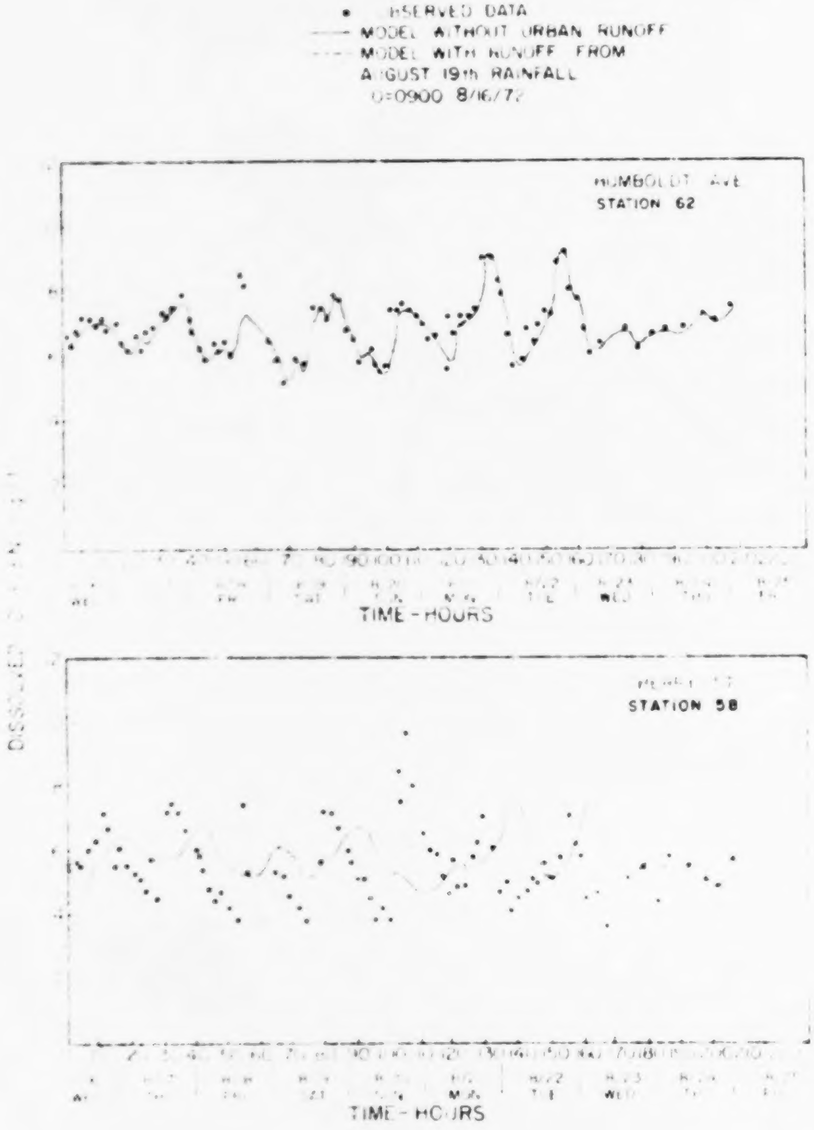


FIGURE 60
OBSERVED VS. CALCULATED DATA
SURVEY IV-HUMBOLDT AVE.,CHERRY ST

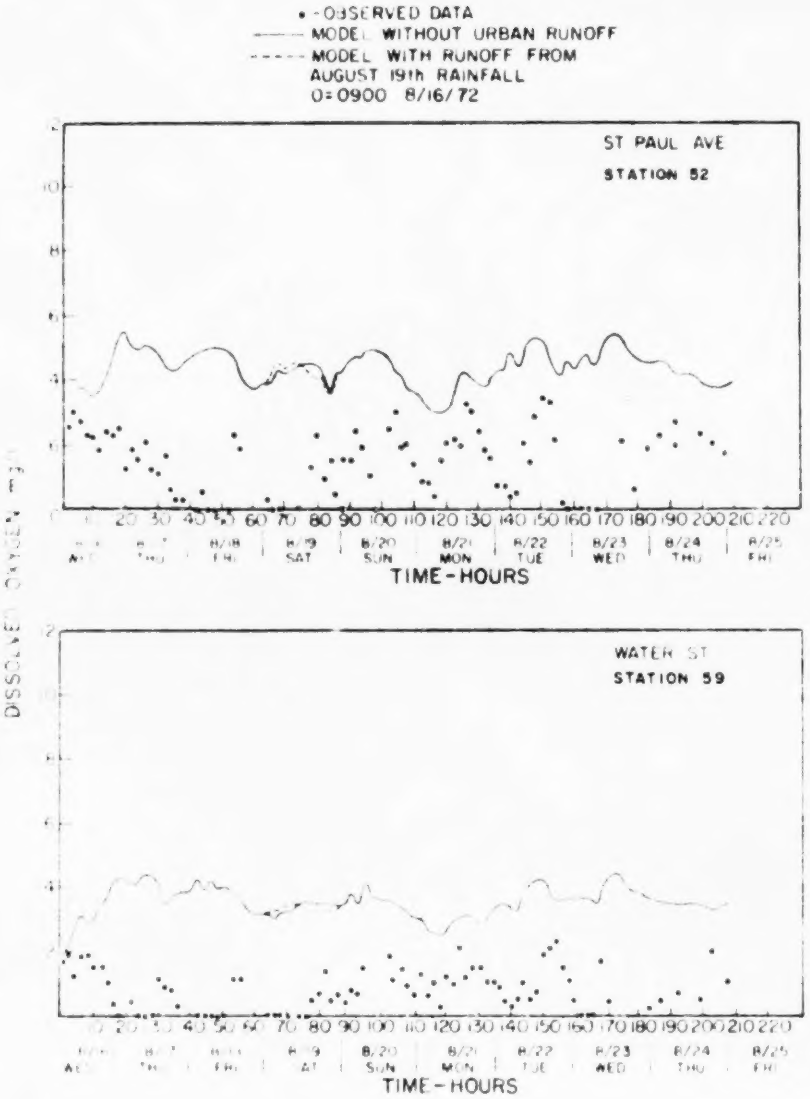


FIGURE [illegible]

OBSERVED VS. CALCULATED DATA
SURVEY IV - ST PAUL AVE., WATER ST.

Table 45. COMPARISON OF DISSOLVED OXYGEN CONCENTRATIONS DURING SURVEYS III AND IV

Survey III (May 17-25, 1972)

Station	DO	DO Deficit
Humboldt Avenue	7.22	1.44
Cherry Street	4.55	4.06
St. Paul Avenue	2.57	6.06
Water Street	2.26	6.39

Average Flow = 270 cfs (at Humboldt)
Average K_d = 0.22/day (segments 4-40)
Average K_a = 0.09/day (segments 1-40)

Survey IV (August 16-24, 1972)

Station	DO	DO Deficit
Humboldt Avenue	6.84	1.43
Cherry Street	5.57	2.61
St. Paul Avenue	1.38	6.56
Water Street	0.76	7.12

Average Flow = 714 cfs (at Humboldt)
Average K_d = 0.25/day (segment 1-40)
Average K_a = 0.22/day (segment 1-40)

deficits would occur during Survey IV. At Cherry Street this is the case and therefore model simulation of both surveys at this station, with a consistent set of parameters is achievable. However, DO deficits observed at St. Paul Avenue and Water Street are lower for Survey III with anaerobic conditions occuring frequently. This indicated the presence and influence of an oxygen demanding source during Survey IV which did not influence the model results during Survey III. Model simulation without the inclusion of these influences would result in the discrepancies encountered. However, it has been demonstrated that the combined sewer overflow occuring during Survey IV was of too short duration to influence or cause any major degradation in DO levels.

Alternative possibilities are suggested. The first is the possibility of bottom scour due to the high velocities induced by the combination of the increased Milwaukee River flow (average greater than 400 cfs) combined with tunnel operation (452 cfs) during Survey IV. Such scouring or stirring of the bottom could result in an increase in BOD in the overlying water column from the suspension of particles and the exposure of a fresh surface which would increase sediment surface oxygen demand. Such a phenomenon would be quite complex and any analysis would be very speculative without further data.

Another alternative possibility would be an increased load due to the Menomonee River. For the model simulation the boundary conditions in the Menomonee were assumed to be 2.5 and 2.0 mg/l for BOD and DO deficit, respectively. The Menomonee River station at the South Second Street Bridge exhibited zero DO levels for 6 of the 9 survey days. This indicates that the Menomonee could contribute somewhat to the deficits observed at the two downstream stations. Quantification of the effects of the Menomonee will be presented in the next section.

The final alternative would be some point source such as a raw waste discharge in the study area. However, no such discharge has been identified.

An additional verification analysis was performed on data obtained during Wet Weather Survey II (September 1970), even though boundary conditions, were less well defined than they were with Survey IV, which has just been discussed. The lack of significant impact from the small storm events in Survey IV, makes it desirable to investigate the effect of a rather substantial storm event which occurred near the end of Survey II. Sufficient confidence in the basic model parameters developed by the verification analysis previously discussed, permitted application of the river quality model to conditions observed during this survey, wherein some assumed values for boundary conditions were necessary.

During Survey II, a major rainfall occurred in which approximately 1" of rain fell, with the bulk of the storm covering a span of about 5 hours, during the morning hours of September 15th. Very brief rainfalls, of small intensity had occurred earlier in this survey. This data has been described in a prior section of the report.

For model simulation of this survey, DO deficit boundary conditions at Humboldt Avenue were based on bi-hourly data. However, water quality measurements other than DO and temperature at Milwaukee River stations were not taken during the survey. Therefore, as with Survey I, other boundary conditions of the model were estimated base, in part, on all water quality data taken during the entire project. A BOD boundary value of 6.5 mg/l was utilized at Humboldt Avenue which is representative of the summer average (refer to Table 8.6, Section VIII). Appropriate BOD dilutions were applied during periods of tunnel operation as previously described. At the Menomonee River boundary, a BOD of 2.5 mg/l was applied except for the period of major runoff on September 15, when a peak value of 6.5 mg/l was utilized. A constant DO deficit of 2.0 mg/l was applied at this boundary. At the Kinnickinnic River boundary, 5.0 mg/l and 10 mg/l were utilized for BOD and DO deficits respectively. DO saturation was assumed at the Lake Michigan boundaries with a background BOD level of 2.5 mg/l. The same dispersion field, kinetic reaction rates and benthal oxygen demands used in the other survey simulation were applied.

The detention tank model was again utilized to evaluate the influent loads and flows into the Milwaukee River based on rainfall data secured from the gauges in the study area. These estimates were scaled to each combined sewer based on drainage area served, and inputed into the model outlet below the North Avenue Dam. The storms occurring on September 13 and 14 were eliminated in the analysis due to their relatively short duration (one hour) and low intensity.

The rate flow regime utilized was based on U.S.G.S. data (refer to Table 43) combined with flushing tunnel operation and urban runoff. During the weekend, when the flushing tunnel was shut down, the average flow in the Milwaukee River above the confluence with the Menomonee was about 160 cfs. An abrupt increase in flow was recorded on September 15 with an estimated peak of 1955 cfs (inclusive of tunnel flows) occuring at 10:00 A.M. During this date the average daily flow in the Menomonee was greater than that of the Milwaukee River as recorded at the U.S.G.S. gaging station. A peak value of 3730 cfs was estimated in the Milwaukee River below the confluence with the Menomonee River.

A comparison of observed versus calculated data is presented in Figures 62 through 65. Overall verification at Cherry Street is good, although the model overestimated the D. O. somewhat during the weekend when the flushing tunnel was shut down. The model does accurately track the general pattern and trend in dissolved oxygen levels under the influence of the flushing tunnel and the substantial surge in flow which occurred on September 15th in association with the substantial storm event. In this verification run, the model projection was extended beyond the termination of observed data, to project the residual influence of the storm event before it is displaced from the area of the river under study.

Verification at St. Paul Avenue and Water Street is good, until the very end of the test run. Both reflect the abrupt increase in DO levels on September 15th. This is attributable to increased reaeration rates associated with the tenfold increase in river flows at that time. As flows return to lower levels following the storm surge, significant deviations are once again observed between observed and calculated data. The discrepancy cannot be explained in terms of BOD concentrations. The combined sewer overflows, which occurred, do in fact result in a significant increase in calculated BOD levels based on model projections. An increase from 5 to 12 mg/l is calculated for Cherry Street; from 3 to 10 mg/l at St. Paul; and from 3 to 8 mg/l at Water Street. However at the high flow rates which prevail, there is insufficient residence time in the River for these reactions to occur, before these combined sewer overflows are flushed from the system. The BOD load from the combined sewer overflows will exert an oxygen demand, but it will occur outside the section of the river being studied.

The close approximation of observed versus projected DO levels, throughout the survey up to the point where high flows were sustained, provides an additional basis for attributing the discrepancy to some phenomenon associated with scour of bottom sediments. At any rate, the implication is again raised that there are other factors at work in addition to those utilized in the model. The same substantial deviation is observed with the higher flow regimes as noted in Survey IV at high flows. The effect of storm discharges cannot account for this deviation.

During Survey II, the water quality at Water Street was somewhat better in terms of DO, than occured at St. Paul. This may be due to the Menomonee River exerting a positive influence at this time, possibly because of higher DO levels in that River. The quality in the Menomonee is more variable and apparently depends to a major degree on storm events, possibly due to its smaller drainage area.

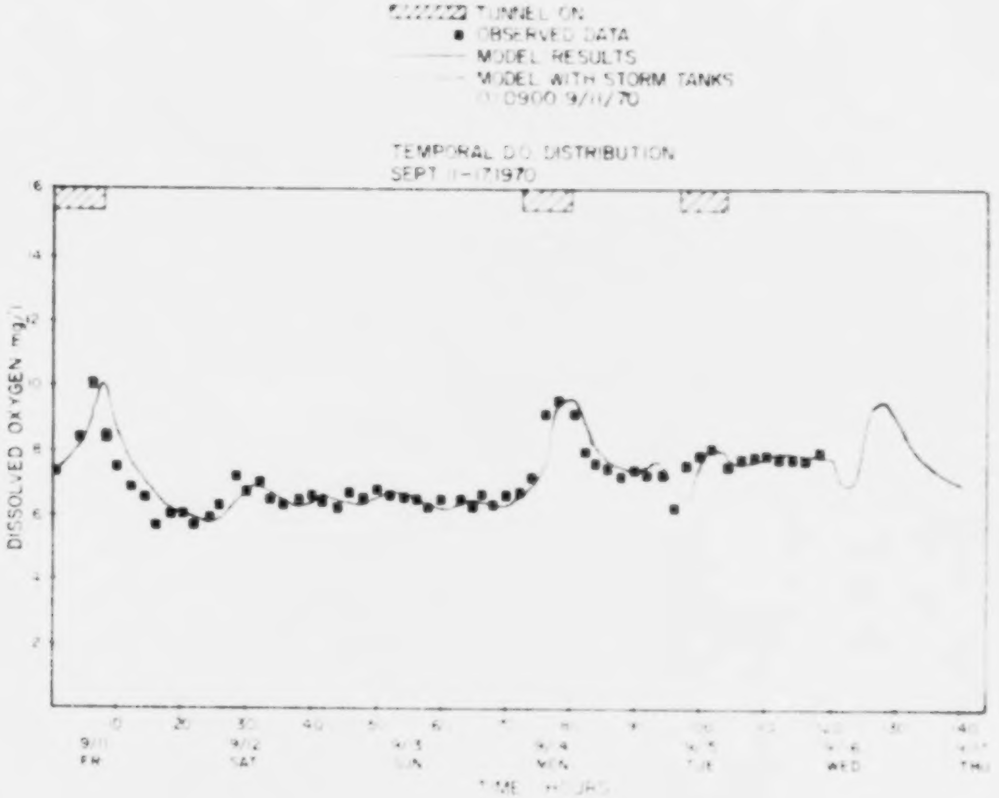


FIGURE 62

SURVEY II – HUMBOLDT AVE

STATION 62

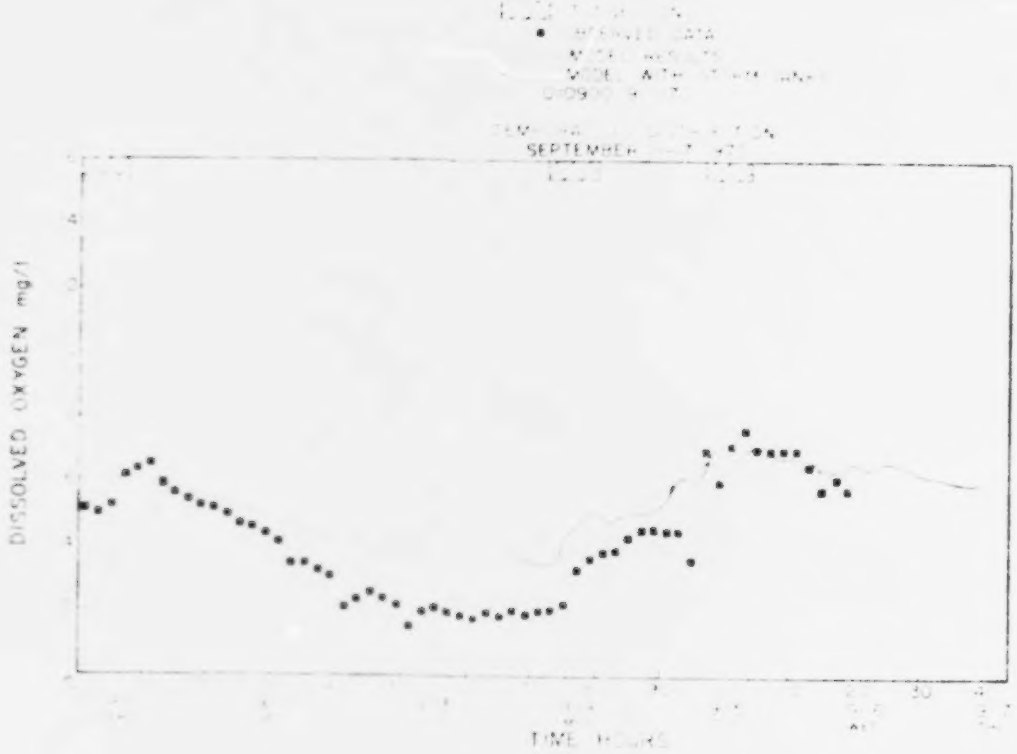


FIGURE 43
SURVEY II - CHERRY ST

STATION 58

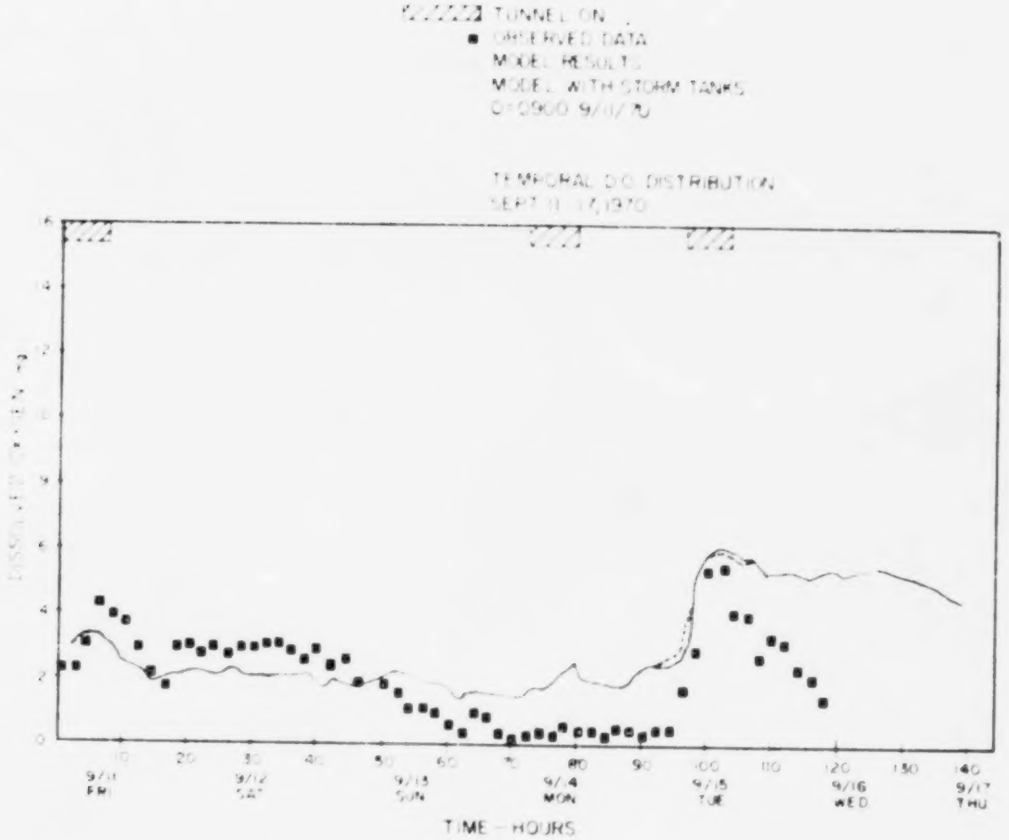


FIGURE 64
SURVEY II - ST PAUL AVE
STATION 52

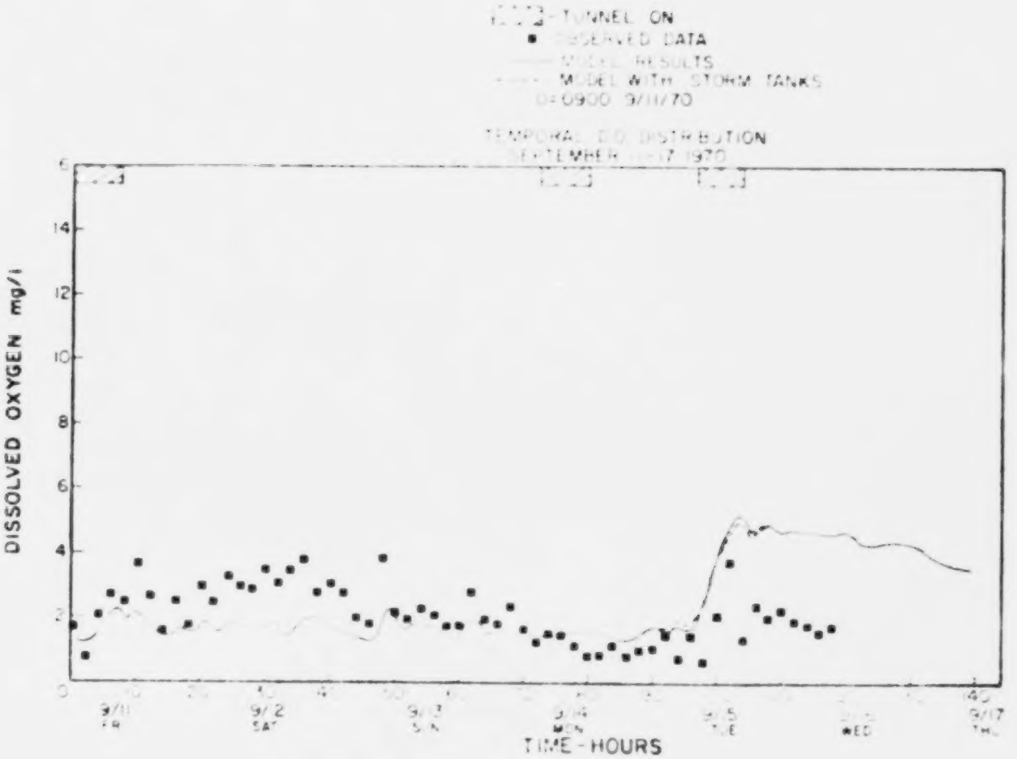


FIGURE 65

SURVEY II - WATER ST

STATION 59

The model projection applied calculated combined sewer overflow loadings over the section of the River between the Dam and the Menomonee River, as has been described. An additional model run was made, assuming that each of the overflow points which contributed a load was equipped with a detention tank of similar unit size (million gallons volume/square mile) to the Humboldt Avenue Demonstration Tank. In this latter case, only the flow and loads which would escape the storm detention system were applied as loads. All overflows were retained except for the last hour of the storm on September 15th when 57% of the hourly rainfall was returned. Table 46 indicates projection storm flow and BOD removed by the project detention tank. Table 47 indicates the scale up of this data to provide loadings to the river from a series of tanks intercepting discharges to the section of the river which was modeled. Output projections from this run are shown on Figures 64 and 65 as dashed lines.

Again, because of the high flows and short residence in the river the effects of the detention system on this section of the river are minimal. Slight reductions in river DO are indicated because of the reduced river flow resulting from detention of combined sewer overflows. However, combined sewer overflows do contribute suspended solids (which over the years adds to the bottom deposit oxygen demand), coliform organisms, BOD, COD, etc. The deleterious effect of these will be greater after the water leaves the river.

TABLE [illegible] MILWAUKEE RIVER - DETENTION TANK
RAINFALL ANALYSIS FOR PROJECT AREA 570 ACRES

[illegible]	Hour	% BOD To Tank	% BOD To River	Flow Rate (cfs) To River	Flow Rate (cfs) To River
[illegible]	[illegible]	[illegible]	[illegible]	2.7	0
[illegible]	[illegible]	[illegible]	0	0	0
[illegible]	[illegible]	[illegible]	0	2.[illegible]	0
[illegible]	[illegible]	[illegible]	0	0	0
[illegible]	[illegible]	[illegible]	0	0	0
[illegible]		[illegible]		[illegible]	0
[illegible]	[illegible]	[illegible]	[illegible]	2.7	0
[illegible]	[illegible]	[illegible]	0	2.7	0
[illegible]	[illegible]	[illegible]	0	2.0	0
[illegible]	[illegible]	[illegible]	0	1.7	0
[illegible]	[illegible]	[illegible]	0	18.9	0
[illegible]	[illegible]	[illegible]	0	44.9	0
[illegible]	[illegible]	[illegible]	0	13.4	0
[illegible]	[illegible]	[illegible]	0	17.0	0
[illegible]	[illegible]	[illegible]	100	42.8	18.4
[illegible]		[illegible]	100	172.3	18.4

Table 47. PROJECTED COMBINED SEWER OVERFLOW LOADS FOR SEPTEMBER 15, 1970 STORM

Model Segment	Drainage Area (Acres)	Drainage Area Scale Factor	Combined Sewer Overflow Loads (lb/hr)	
Upstream	290	Drainage Area of Tank 5.09	Without Tanks	With Tanks
4	719	1.26	1055	378
10	307	0.539	451	162
16 16	408	0.716	599	215
20	407	0.714	598	214
23	95	0.167	140	50
24	53	0.093	78	28
26	96	0.168	141	50
28	118	0.207	173	62
29	96	0.168	141	50
31	134	0.235	197	71
34	9	0.016	13	5
35	67	0.118	99	35
37	93	0.163	136	49
38	37	0.065	54	20
40	9	0.016	13	5
41	35	0.061	51	18

Loads Applied to River at Model Time 25.01 to 26.00 Hours

SECTION X

COMBINED SEWER OVERFLOW DETENTION TANK APPLICATION AND MILWAUKEE RIVER QUALITY

DISCUSSION OF DETENTION TANK CAPABILITIES IN CONTROL OF COMBINED SEWER OVERFLOW

Combined sewer overflow detention tanks have been shown by this program to be effective in preventing a large proportion of the contaminants found in combined sewage from entering receiving waters. Removal achieved is directly related to tank size in relation to the drainage area served. Removal efficiencies will vary significantly with individual storm events and will be particularly influenced by storm size and pattern, and by whether there has been sufficient time to empty the tank from the previous storm event. However, when performance over a year's time is compared an evaluation of longer range effectiveness can be made.

Removals for BOD and Suspended Solids can range from approximately 30% to in excess of 80% as tank size is increased from 1 to 6 million gallons volume per square mile of drainage area. The removal efficiency is slightly better in dry years compared with wet years for all sizes, and the differential between performance in wet verses dry years increases with tank size. These relationships are illustrated by Figure 66.

Figure 67 illustrates the decreasing efficiency per unit volume as tank size increases. Tank sizes of 1 MG/ sq mi show a unit efficiency (% Removal/MG/Sq Mi) of about 30% for both BOD and COD. This unit efficiency decreases to approximately 15% removed per unit volume for a tank providing a volume of 6 MG per square mile of drainage area.

An analysis of the volumetric efficiency of the combined sewer overflow detention tank, defined as the quantity of combined sewer overflow retained by the tank, to the total quantity discharged to it, has been made. Figure 68 illustrates the effect of tank size on the

FIGURE 66
STORM WATER BOD AND SUSPENDED SOLIDS REMOVAL AS A FUNCTION OF TANK SIZE

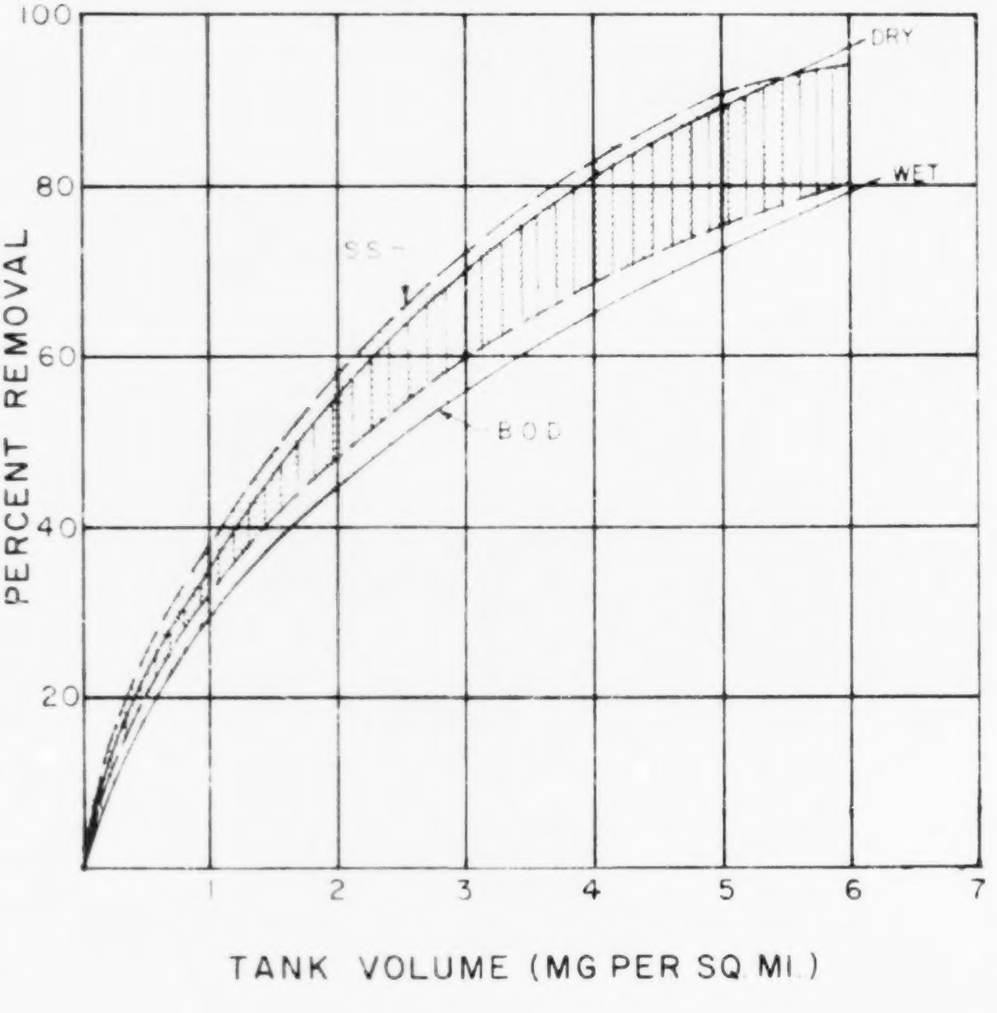


FIGURE 67

UNIT SIZE REMOVAL EFFICIENCIES
FOR COMBINED SEWER OVERFLOW
DETENTION TANKS

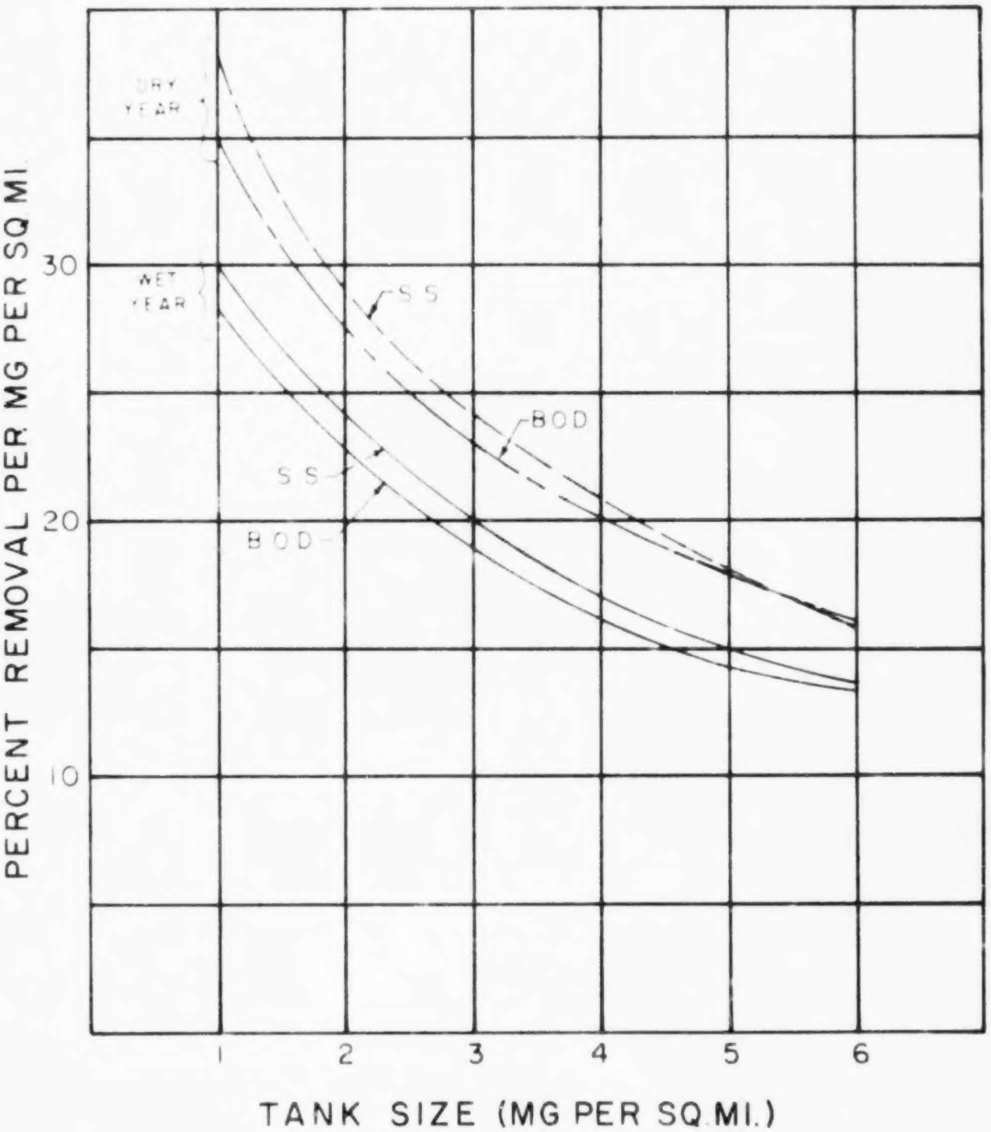
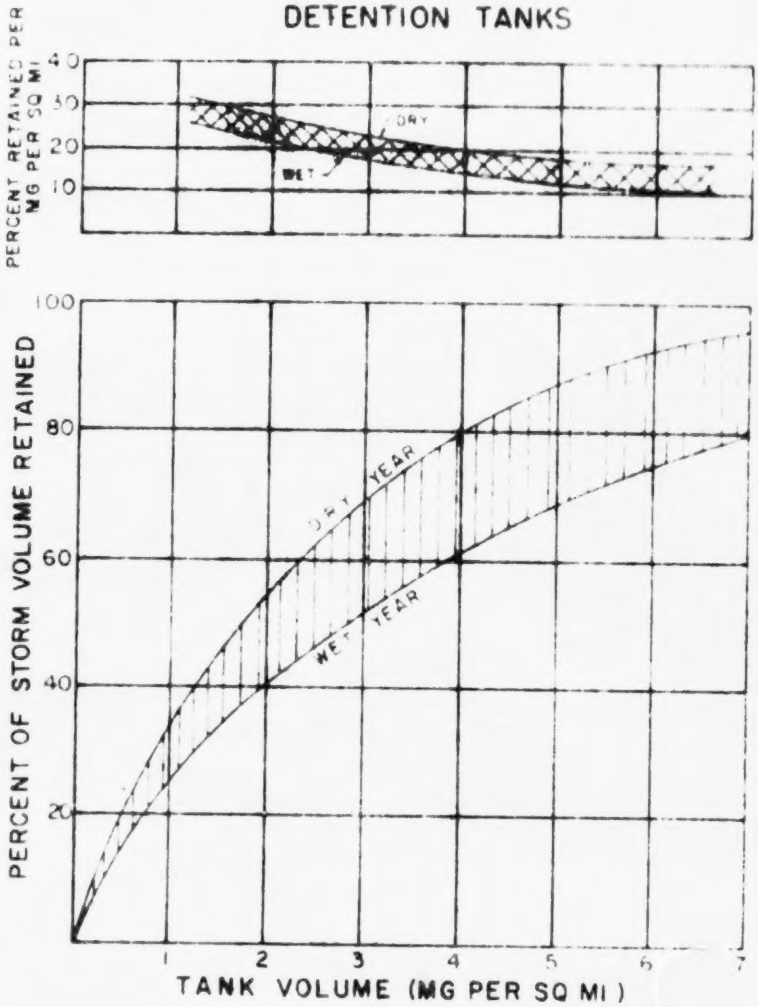


FIGURE 68
VOLUMETRIC EFFICIENCY FOR COMBINED SEWER OVERFLOW DETENTION TANKS



total interception and detention of storm water overflows. The removal of soluable contaminants present in combined sewer overflows will be directly related to the interception and retention of storm water overflows which carry them. Thus, the volumetric efficiency of the detention tank shown by this figure also represents the removal efficiency for all contaminants not subject to additional removal by sedimentation of chlorination.

A comparison of volumetric efficiency with total removal efficiency for BOD and Suspended Solids provides an instructive insight into the relative significance of sedimentation as a supplement to retention in the overall performance of a combined sewer overflow detention tank. Figure 69 plots calculated removal efficiency vs. tank size based on rainfall data from a year of normal precipitation at Milwaukee (Year 1970 - 28.85" rainfall).

The combined sewer overflow detention tank model calculates only very minor additional removals of BOD and Suspended Solids by sedimentation, compared with the amounts removed strictly by virtue of the retention of storm overflows. A similar comparison for a wet and a dry year is presented by Figures 70 and 71. All comparisons shown only a relatively small increase in removal of BOD and Suspended Solids compared with removal due to retention. The greatest effect of sedimentation occurs during a wet year (Figure 70) where removals are increased an additional 5% or so.

It may be concluded that some advantage exists in designing a detention tank with a layout which permits effective sedimentation to occur. However, where such design considerations impose a significant additional cost factor, economic justification may not be present. Where space or construction constraints dictate, tank shapes which do not lend themselves to effective sedimentation efficiency can be validly considered because of such a tank's value in retaining pollutant flow. The major consideration in designing combined overflow detention tanks is volume and cost effectiveness considerations should concentrate on maximum volume which can be achieved at a given cost.

The combined sewer overflow detention tank design provides for chlorination of all water which overflows the tank and discharges to the river. Effectiveness of this operation in the destruction of total and fecal coliform bacteria will be related to chlorine dosage applied and the chlorine demand of the waste. Two additional factors will be of major influence on the effectiveness of this operation. The wide variability in coliform counts in the combined sewerage is important since the kill effected is in terms of percentage reduction of those

FIGURE 69
CONTAMINANT REMOVAL
VS
VOLUMETRIC EFFICIENCY
NORMAL YEAR
1970
28.85" RAIN

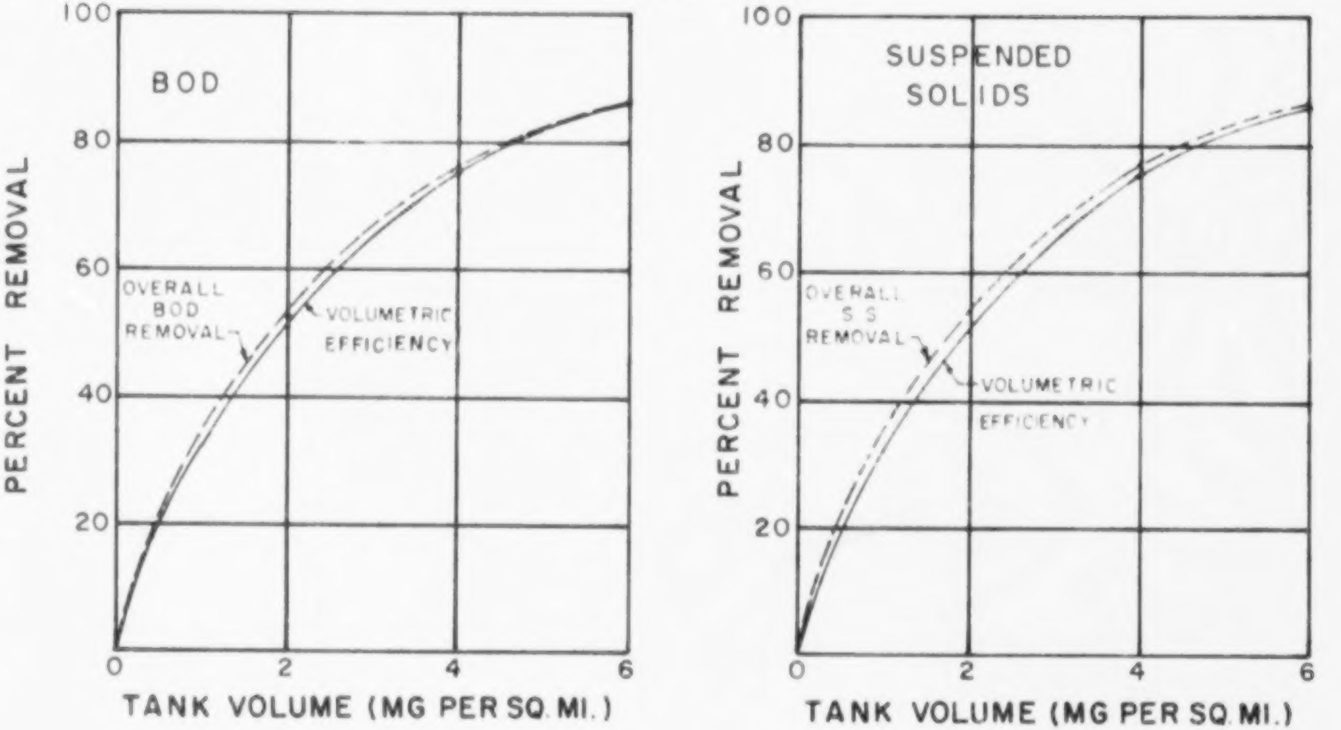


FIGURE 70
COMPARISON
CONTAMINANT RETENTION
VS
VOLUMETRIC RETENTION
WET YEAR

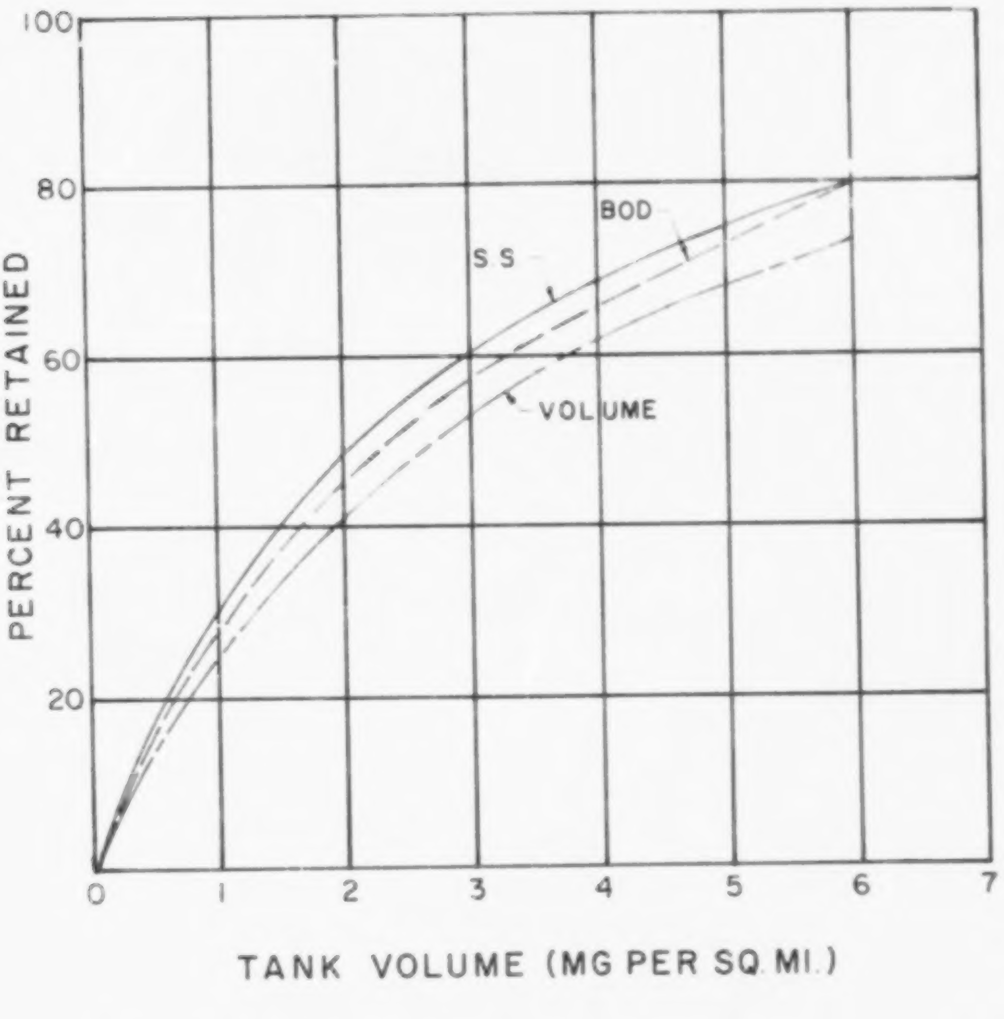
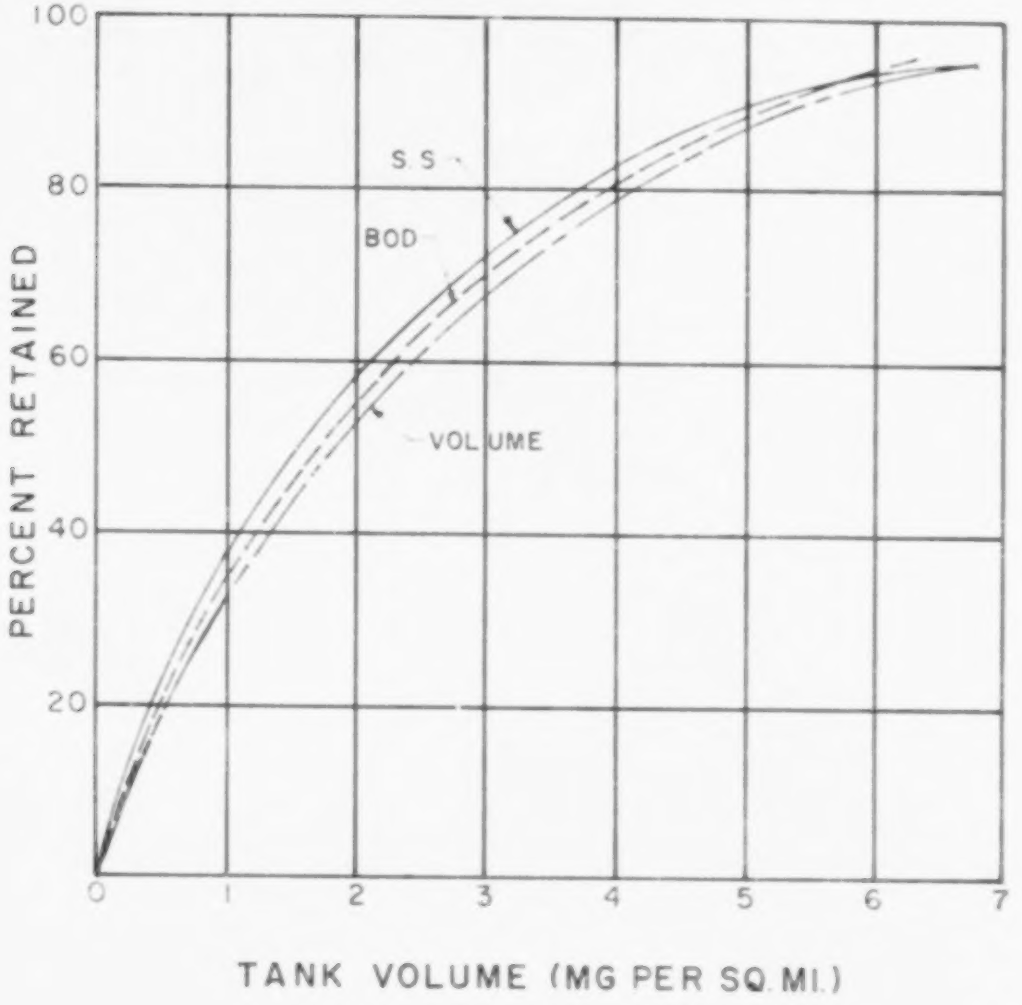


FIGURE 71
COMPARISON
CONTAMINANT RETENTION
VS
VOLUMETRIC RETENTION
DRY YEAR



organisms initially present.

Flow rates fluctuate very rapidly and over very wide ranges during some storm events, such that contact times will often be very short.

From the data available, it is not possible to provide adequate definition of this aspect of tank performance. Quality data for tank overflows record total coliform counts ranging from essentially zero, to several million per 100 ml. Fecal counts behaved similarly, though with lower total numbers. The primary difficulty preventing a meaningful analysis of coliform data, was the deficiency in flow rate data during the times when the detention tank was overflowing in the initial stages of testing resulting from deficiencies in flow measuring equipment.

A preliminary assessment was made utilizing overflow quality data, and the output of the storm tank model to provide the order of magnitude of the flow rate at times when the various coliform counts were observed. The results of this analysis, which must be considered a relatively low order approximation only, are presented in Figure 73. This plot compares coliform counts in the final discharge from the tank with the throughput rate expressed in terms of million gallons/hour.

FACTORS INFLUENCING WATER QUALITY IN THE MILWAUKEE RIVER

Analysis of long term quality data for the Milwaukee River described in an earlier section of this report, indicated that water quality in the river section downstream of the North Avenue Dam is influenced to a substantial degree by conditions in the drainage basin upstream of the test area. The intensive surveys and the analysis by mathematical model have made it possible to investigate the impact of factors which make their presence felt downstream of the dam. In this analysis, upstream influences in effect isolated by the technique of establishing the quality of water entering the area of interest, along with its variations, as a boundary condition for the model. The effects of the superimposed local conditions can then be identified and quantified. These above mentioned factors are of course variable with time.

The verification analysis which has been presented in the previous section has indicated by the match achieved between predicted and observed data, that the river quality model provides an accurate quantification of the major influences on dissolved oxygen levels. A good verification was achieved for dry weather data surveys under different flow rate regimes and flushing tunnel operation schedules. Wet weather survey data verified the model as far downstream as Cherry Street. The significant divergence between observed and predicted response at the lower end of the river (St. Paul Avenue and South

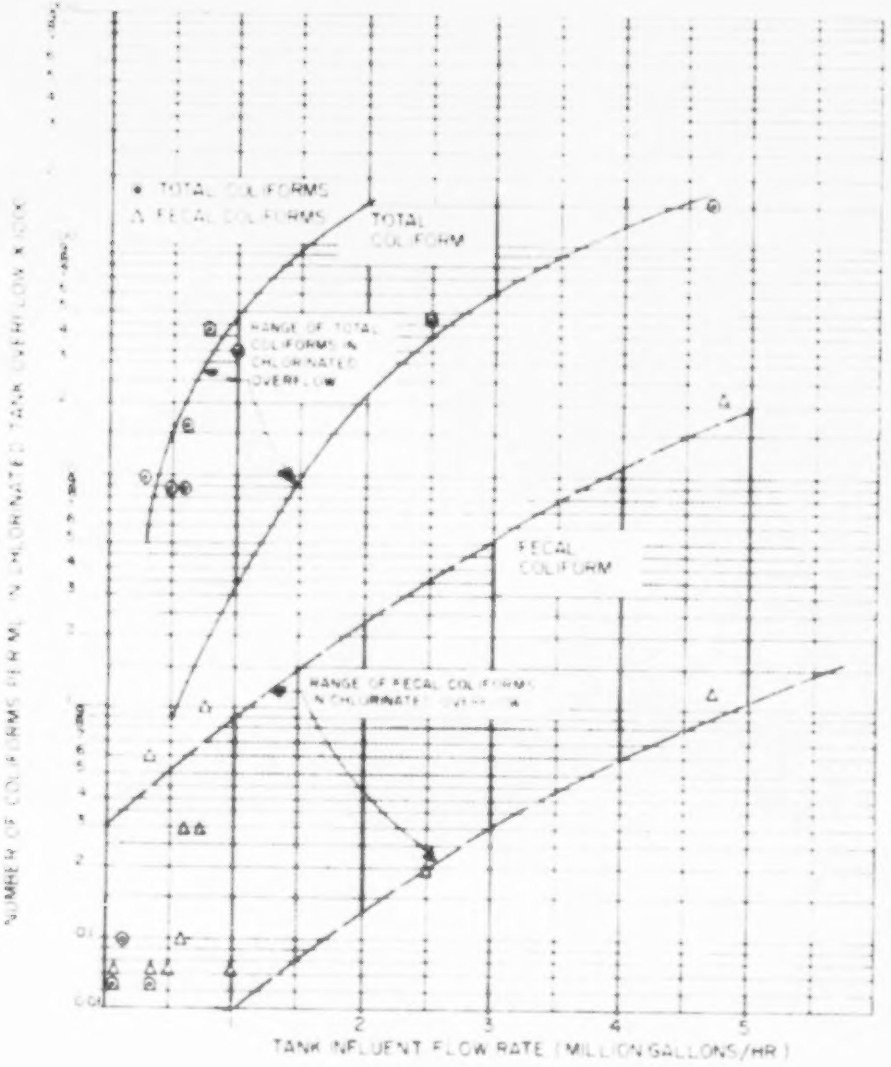


FIGURE 72
COLIFORM COUNT IN DETENTION TANK OVERFLOW
VS
TANK THROUGHPUT RATE

Water Street, is attributed not to inadequacy in the basic quality model, but rather to external influences which have not been identified previously, and which the model did not incorporate.

From general observations made during the course of the project, and particularly from a review of quality data obtained during the study, the following factors have been identified as exerting significant influence on water quality in the section of the river downstream of the dam:

a) Upstream quality variations in response to loadings, storm runoff, and flow changes.
b) Significant algal and macrophyte activity, occuring at times both above the dam, and in downstream sections.
c) Flushing tunnel operation.
d) Benthal oxygen demand from accumulated sediments.
e) The Menomonee River.
f) Urban Storm runoff.
g) Combined sewer overflow

The river quality model has been utilized in performing a series of application runs to study the significance of these factors. The results of these application studies are discussed below:

Upstream Conditions

Both observed data from long term river analysis and intensive surveys, as well as model results indicate that upstream conditions provide a major load input to the area of the river under investigation. Relatively high levels on contamination exist in the Milwaukee River before its waters reach the study area. At times when oxygen resources are high and BOD levels are low in this incoming water, better quality conditions will prevail in the test area. Total river flow has a major influence on oxygen resources both because of increased reaeration rates at high velocity, and because of reduced residence time in the test area for oxygen demanding substances to exert their full effect. As an example of the magnitude of this impact, it has been calculated that water quality conditions in the water reaching Humboldt Avenue accounted for approximately 59 percent, 33 percent, and 2[illegible] percent of the oxygen deficits at Cherry Street, St. Paul Avenue and Water Street, respectively, during dry weather Survey I.

Photosynthetic Activity

Activity by algae and macrophytes is at times intense. Diurnal fluctuation in dissolved oxygen in the order of 7 mg/l were observed in May 1972 at the North Avenue Bridge, upstream of the dam. These variations, as well as the algal cells are transported into the test section of the river and at such times significantly influence oxygen levels. Daily variations in dissolved oxygen of about 3 mg/l occur in the lower reaches of the river, such that oxygen levels are depressed to or close to zero at times when concentrations would otherwise be several mg/l higher.

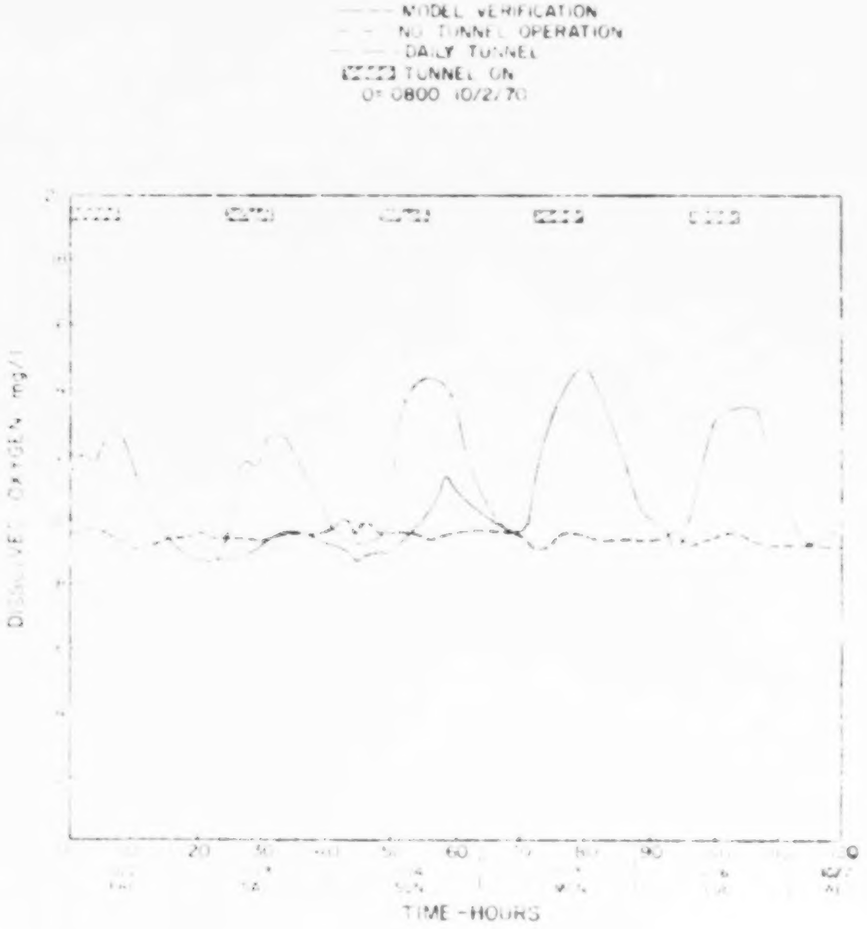
Flushing Tunnel Operation

The flushing tunnel is a controllable element, and one which exerts a significant influence on quality.

The model verification for Survey I was used as a basis of comparison to identify the effect of flushing tunnel operation. During Survey I the tunnel was operated three out of the five survey days. Two alternate tunnel operation schemes were studied using the water quality model. This necessitated a reevaluation of boundary conditions at Humboldt Avenue and appropriate changes in the flow regime.

For the first scheme, it was assumed that the tunnel was not operated at all during the survey period. DO deficit at Humboldt Avenue was assumed constant and equal to the average of the data observed at Humboldt Avenue during the weekend for Survey I when the tunnel was not in operation. A BOD boundary condition of 5 mg/l was assumed at Humboldt Avenue. The second scheme approximated the effect of daily operation (7 day operation) of the tunnel for 7.5 hour (0800 - 1530). Boundary conditions at Humboldt Avenue during the weekend were modified to reflect this daily tunnel operation based on data observed during those days when the tunnel was operated. All other input parameters to the model were the same as those applied for the original verification run. The effectiveness of flushing tunnel operation can be readily seen by comparison of the calculated profiles at the four key Milwaukee River stations as presented in Figures 73 through 76. It is estimated that tunnel operation for 7.5 hours per day every day improves DO levels by about 3.5, 2.0, and 1.5 mg/l at Cherry Street, St. Paul Avenue, and Water Street, respectively. Also note the calculated improvement in DO levels as projected under daily tunnel operation as opposed to weekend shut-down as was predicted during Survey I.

The favorable influence of the flushing tunnel is the result of both the additional oxygen resources introduced by virtue of the high DO content of the water delivered by the tunnel, and also because of the higher atmospheric reaeration rate in the river, which the tunnel operation induces because of increased velocities. However, our

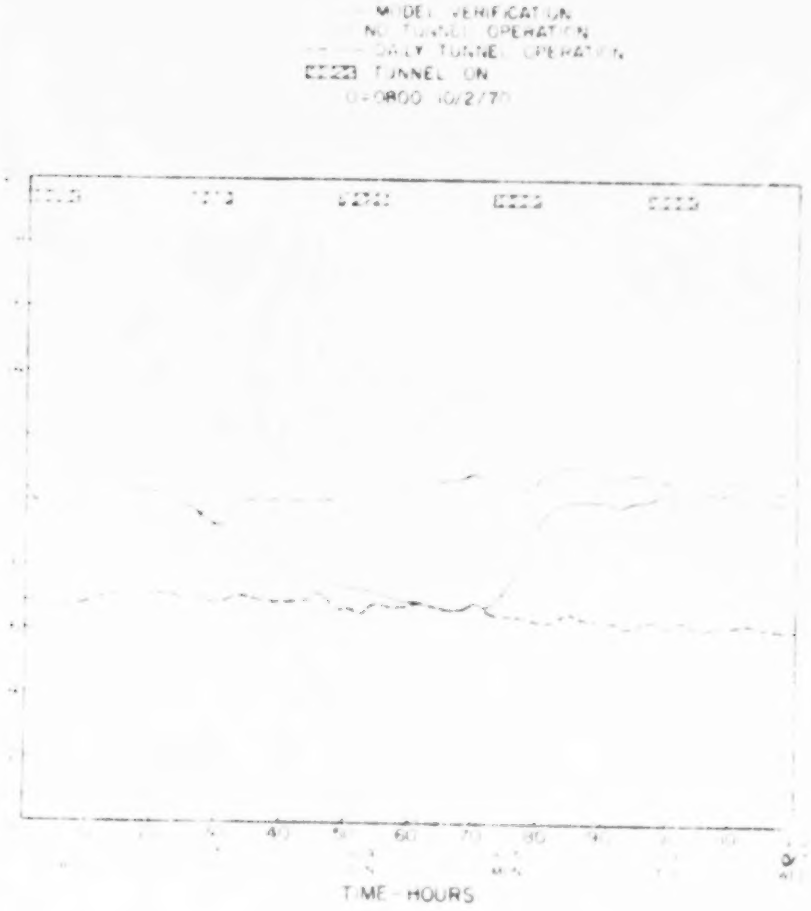


Note: The significant Deviation between the Daily Tunnel Curve and the Model Verification Curve are due to the fact that the tunnel was only actually operated 1 of the 5 days of Survey I.

FIGURE 73

MODEL APPLICATION AT
HUMBOLDT AVE-SURVEY I

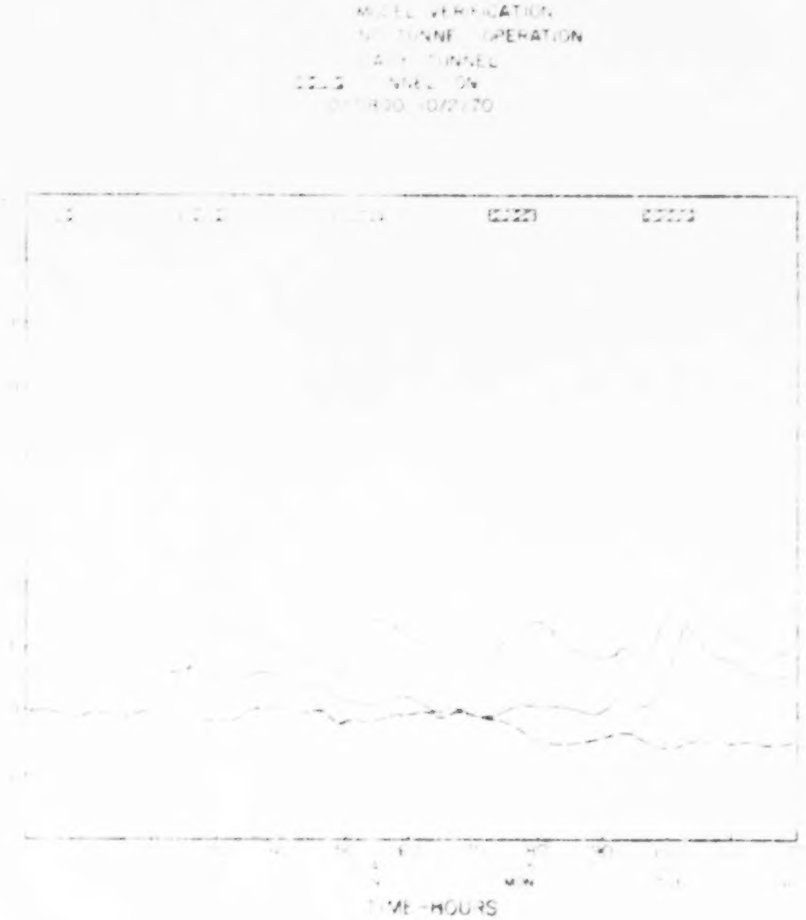
STATION 62



Note: The significant Deviation between the Daily Tunnel Curve and the Model Verification Curve are due to the fact that the tunnel was only actually operated [illegible] of the 5 days of Survey I.

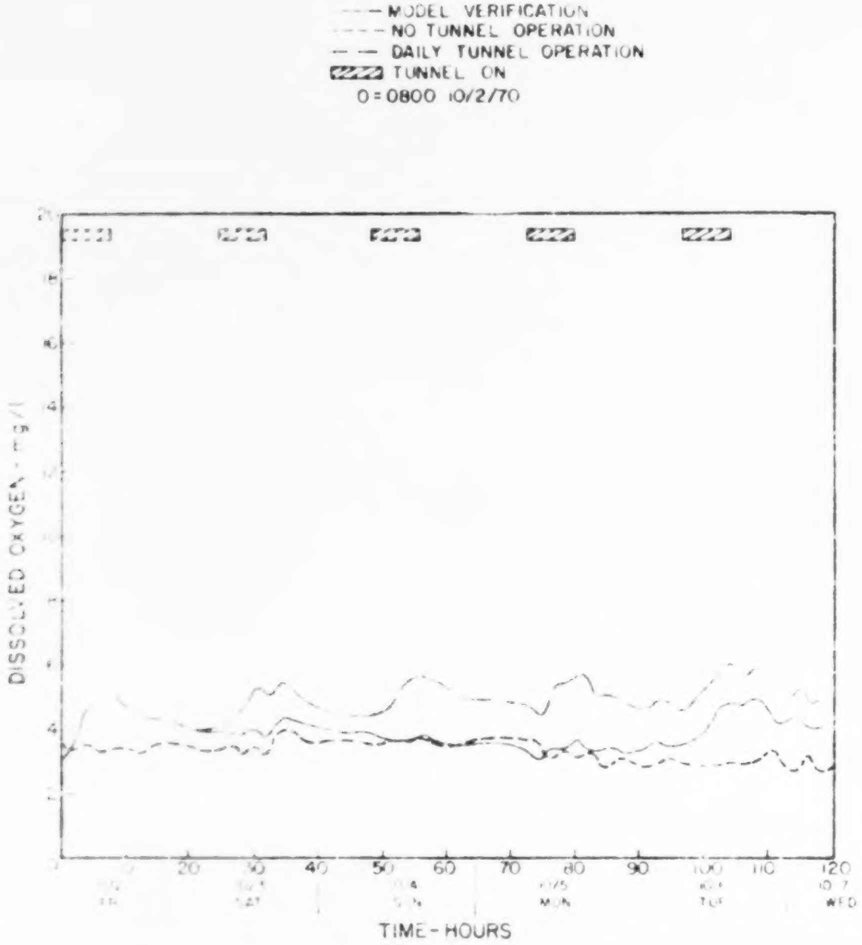
FIGURE 74
MODEL APPLICATION AT
CHERRY ST. – SURVEY I

STATION 58



Note: The significant deviation between the Daily Tunnel Curve and the Model Verification Curve are due to the fact that the tunnel was only actually operated [illegible] of the [illegible] days of Survey I.

FIGURE 75
MODEL APPLICATION AT
ST PAUL AVE-SURVEY I
STATION 52



Note: The significant deviation between the Daily Tunnel Curve and the Model Verification Curve are due to the fact that the tunnel was only actually operated 3 of the 5 days of Survey I.

FIGURE 76

MODEL APPLICATION AT WATER ST. — SURVEY I STATION 59 analysis suggests that indiscriminate tunnel operation may have an adverse effect at times. During Survey IV for example the flushing tunnel was operated 16 hours/day on all days at a rate of 425 cfs. River flows, as recorded above the North Avenue Dam, averaged greater than 400 cfs. Total flows in the river below the dam therefore exceeded 825 cfs with sustained peaks of more than 1,000 cfs. Comparison of the data of Survey III (Average flow equals 270 cfs) with that of Survey IV (refer to Table 45) show that higher DO levels existed for Survey III at St. Paul Avenue and Water Street. As described in the wet weather verification section this indicated introduction of additional sources of DO deficit during Survey IV. A distinct possibility is that sufficiently high velocities, either by scouring or stirring up bottom sediments, increase oxygen requirements associated with bottom sediments. Therefore, the beneficial effect of tunnel operation under high flow conditions is somewhat questionable under present conditions. Additional investigation would be required before a definite conclusion can be made.

Sludge Deposits

Visual observations of active gasification, as well as model verification using the relatively high benthal oxygen demand of 4 mg/m²/day, confirms the significance of accummulated sediments on water quality. An analysis was performed to define the effect of the elimination of bottom sludges. The verification of Survey III was used as a basis of comparison. The benthal oxygen demand of 4 gm/m^2/day was eliminated from all river segments in the model. All other model parameters were unchanged. The projected improvement in DO levels under the conditions of Survey III can be seen in Figures 77 and 78. The elimination of bottom deposits has increased impact as one proceeds downstream with an improvement of approximately 1.8 mg/l calculated at Water street. Under lower flow conditions the elimination of the estimated benthal demand would result in an even greater improvement in DO quality. Figure 79 presents a plot of the estimated DO deficit at Water Street due to a bottom demand in all river segments of 4 gm/m²/day for a range of the average flows observed in the study. This bottom demand may increase if scouring action or other disturbance occurs. These results are based on a steady state analysis.

The Menomonee River

The Menomonee River has a variable influence on the Milwaukee estuary. Its impact may be relatively slight during dry weather periods with low flows and reasonably good quality. Under at least some wet weather conditions, its influence is very significant.

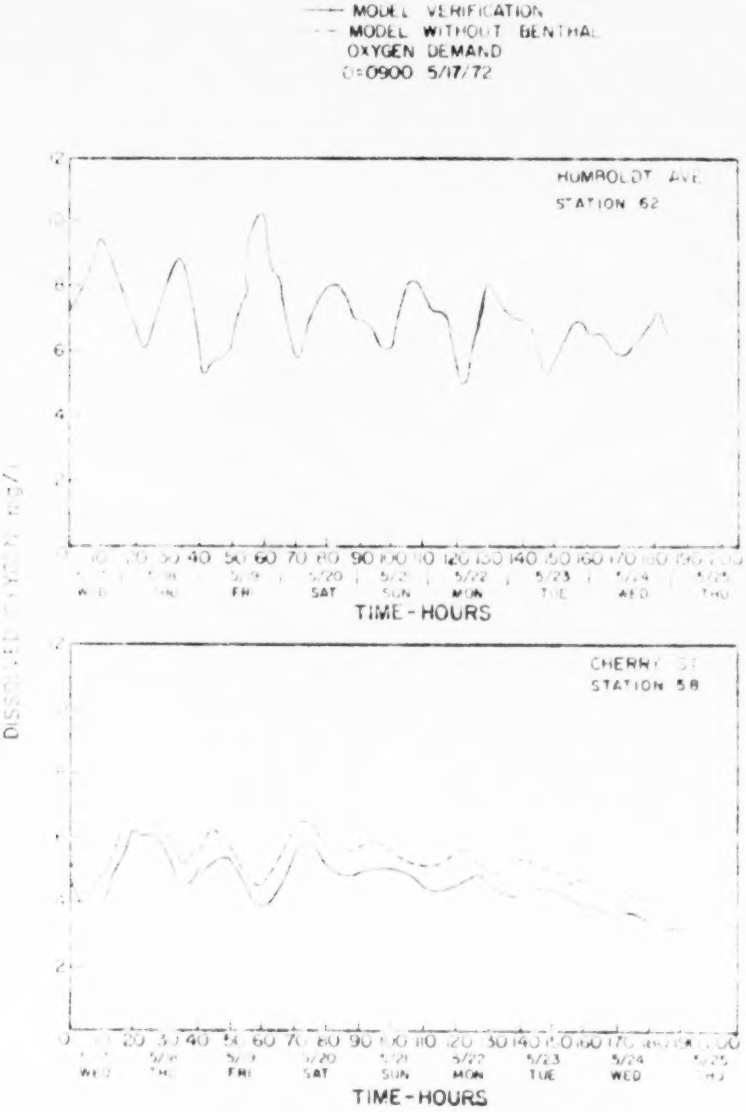


FIGURE 77
MODEL APPLICATION AT
HUMBOLDT AVE. & CHERRY ST.-SURVEY III

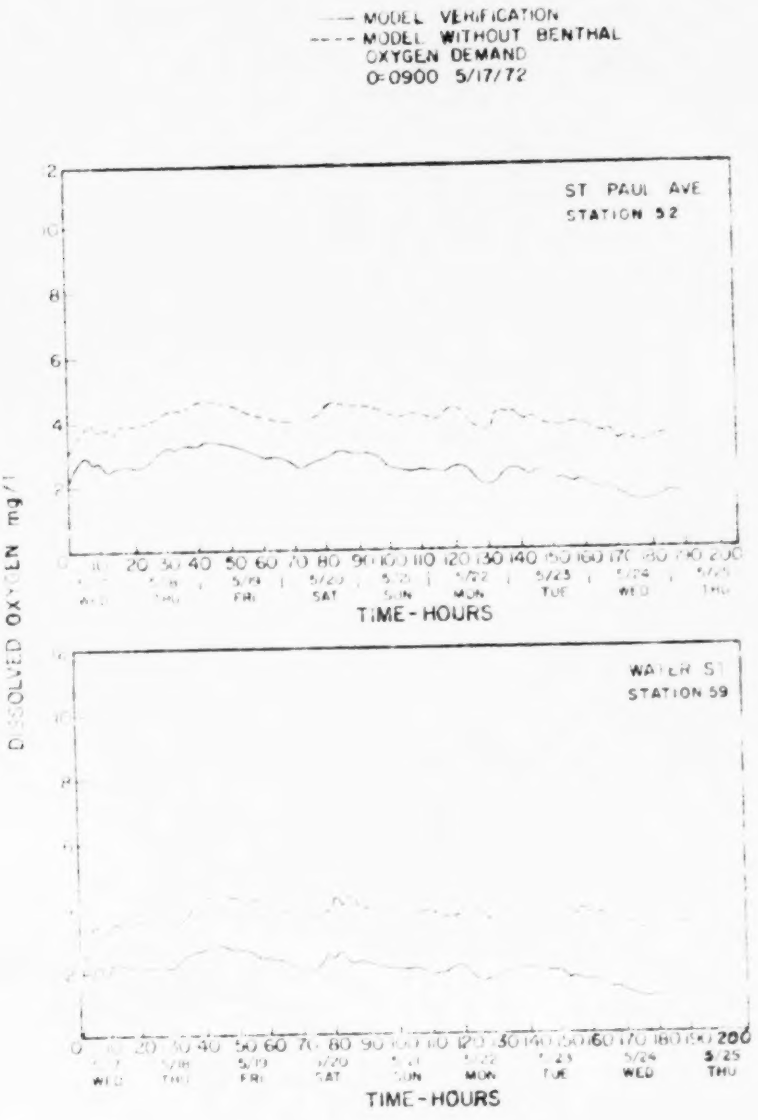


FIGURE 78
MODEL APPLICATION AT
ST PAUL AVE. & WATER ST.-SURVEY III

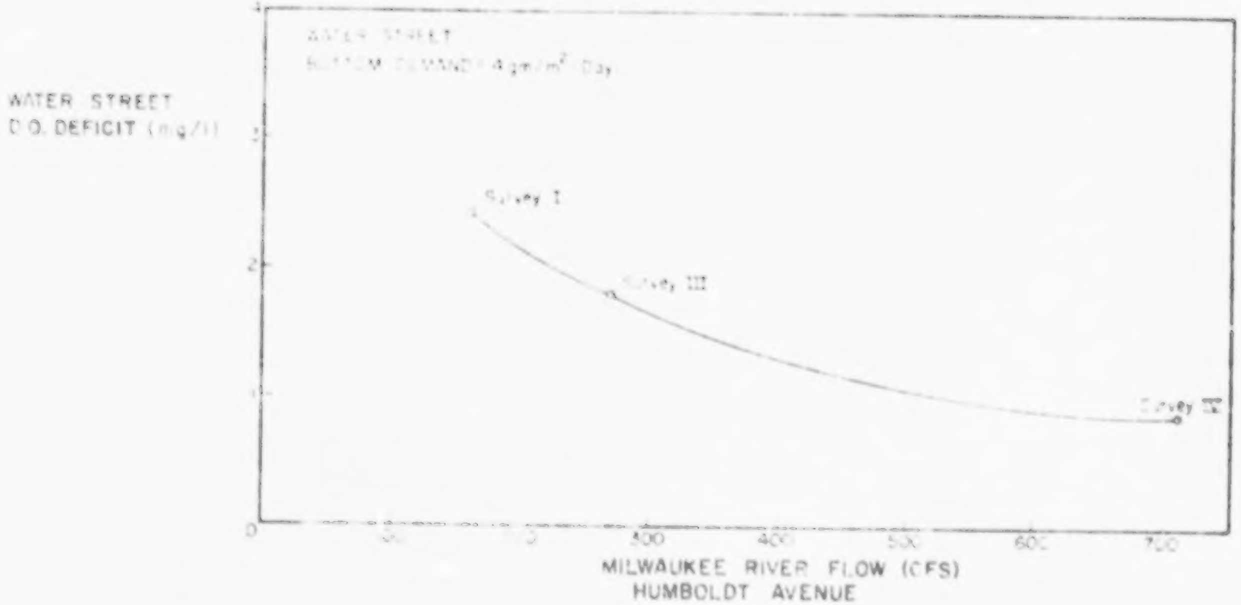


FIGURE 79

DISSOLVED OXYGEN DEFICIT DUE TO BENTHAL OXYGEN DEMAND

For example, during each of the wet weather intensive surveys, flows in the Menomonee River reached very high levels on one or more days. Mean daily flows as high as 400 and 500 cfs were recorded on individual days, providing a total flow of the same magnitude as Milwaukee River flows at those times. Further, extremely poor quality was observed, with zero dissolved oxygen levels. Under such conditions, it is apparent that poor quality in the Menomonee River will result in degradation of quality in the Milwaukee River in the vicinity of St. Paul Avenue and Water Street.

A steady-state model analysis of the Milwaukee River was made using data developed during the various intensive surveys. Such an analysis permits investigation of unit responses to various loading factors. In this particular analysis the effect of the Menomonee River on oxygen deficit at the test stations on the Milwaukee River was calculated for each of the flow regimes which prevailed during the particular surveys. In Table 46, calculated oxygen deficits in the Milwaukee River due solely to the loading imposed by the Menomonee are listed. The deficit response shown is based on an assumed BOD concentration of 15 mg/l and oxygen deficit of 10 mg/l in the Menomonee. This comparison is intended to illustrate the magnitude of the influence under the assumed quality conditions in the Menomonee. At times of higher quality in the Menomonee, the resulting deficits in the Milwaukee would be proportionately less.

The analysis illustrates both the complex nature of the quality response and the magnitude of the impact which could occur. The Menomonee has little or no effect on oxygen deficiency at Humboldt Avenue or Cherry Street, particularly during higher river flows. However, it can influence oxygen deficits in the vicinity of St. Paul Avenue and Water Street by as much as 1 to 2 mg/l even under the lower flow conditions.

Combined Sewer Overflow

The effect of combined sewer water overflows in the section of the Milwaukee River which was studied, is relatively complex. Although BOD loads discharged to the river in this manner will tend to reduce the dissolved oxygen level, there are so many other significant influences on quality in the river section between the North Avenue Dam and the outlet at Lake Michigan that overflows which directly discharge into this section of the river do not have a major effect on dissolved oxygen levels in the River.

Analysis of wet weather survey data and model projections indicate that small overflows have a barely detectable effect. When model

Table 48. CALCULATED RESPONSE OF MILWAUKEE RIVER TO MENOMONEE RIVER

Survey - October, 1970 (Dry Weather)

Milwaukee River Flow - 160 cfs
Menomonee River Flow - 24 cfs (15 percent of Milwaukee River)
Calculated Dissolved Oxygen Deficit in Milwaukee River from
Menomonee River DO Deficits in mg/l

At	Menomonee River BOD (1) mg/l	Menomonee River Deficit (1) mg/l	Total DO Deficit for Milwaukee River Due To Menomonee River Influence
Humboldt	0.0	0.00	0.0
Cherry	0.04	0.07	0.11
St. Paul	[illegible]	0.89	[illegible]
Water	0.73	0.99	1.72

Survey - May, 1972 (Dry Weather)

Milwaukee River Flow - 290 cfs
Menomonee River Flow - 35 cfs (12 percent of Milwaukee River)
Calculated Dissolved oxygen deficit in the Milwaukee River from
Menomonee River BOD and Deficit:
DO Deficits in mg/l

Humboldt	0.0	0.0	0.0
Cherry	0.01	0.01	0.02
St. Paul	0.38	0.71	1.09
Water	0.51	0.95	1.46

Survey - August, 1972 (Wet Weather)

Milwaukee River Flow - 714 cfs
Menomonee River Flow - 107 cfs (15 percent Milwaukee River)
Calculated Dissolved Oxygen Deficit in the Milwaukee River from
Menomonee River BOD and Deficit:
DO Deficits in mg/l

Humboldt	0.0	0.0	0.0
Cherry	0.0	0.0	0.0
St. Paul	0.34	0.54	0.88
Water	0.74	1.19	1.93

projections are made with and without the combined sewer overflow loads entering the river. During the major storm event which occurred during Survey II, the potential effect on dissolved oxygen concentrations from the large increases in BOD, was compensated for by the large flow increases which accompanied the storm. The higher flows exert a dual effect of increased reaeration rates and higher flush out rates. In fact model projections indicate that for this particular set of events, detention of combined sewer overflows by a system of tanks will actually result in a smaller DO value in the river. This effect is due to the reduction in river flow rate due to the volume of storm water detained.

Under prevailing conditions, combined sewer overflow into the river between the Dam and the Menomonee River does not for the most part significantly degrade dissolved oxygen in the immediate area. There may be some particular combination of events, storm size, and pattern, river flow, etc., where a substantial effect would be observed, but this will not be common. Combined sewer overflows will contribute coliform organisms, BOD, COD, etc., however, their deleterious effect will be felt after the water leaves the River and enters Lake Michigan. Combined sewer overflows will additionally contribute a considerable amount of suspended solids which will contribute to the increase of bottom sediments, which create benthal demand.

There seems to be little question that combined sewer overflows are largely responsible for the degraded conditions observed in this section of the Milwaukee River. Overflows have contributed over the years to the high benthal demand observed, and certainly contribute prominently to the water quality in the Menomonee River and in the Milwaukee River above the study area. All of these factors have been shown to have substantial effects on quality in the section of the river under study.

DETENTION TANK APPLICATION - CITY OF MILWAUKEE

Abatement of discharge of combined sewer overflows for the entire City of Milwaukee is a highly complex problem. The solution could involve the construction of detention tanks, similar to the Humboldt Avenue facility, at many locations. However, a comprehensive plan for elimination of such overflows will require the evaluation of other methods, and the ultimate plan may incorporate a variety of measures, selected for suitability according to the magnitude and nature of the problem in various parts of the City.

Although detention tanks have not necessarily been established as the best method of eliminating combined sewer overflows in every case, the results of this study indicate that they will be a viable and economical tool. For purposes of demonstrating the cost impact of the problem, and to facilitate comparison with other methods of abatement, a cost estimate has been developed for construction of thirteen detention tanks to receive flows from all combined sewer overflow points on the Milwaukee River in the City. These detention tanks will receive the combined sewer overflow from the approximately 5800 acres tributary to the Milwaukee River in the City or about 9 of the 27 square miles of the combined sewer area in the City.

Based on estimated December, 1971 price indexes, the approximate preliminary cost estimate for this construction is $29,5000,000. This cost includes the construction of 13 detention tanks, with their associated control buildings and equipment as indicated on Table 49. The estimate does not include costs for land, right-of-way, sewer construction, contingencies or additional treatment facilities which would also add a considerable amount to the cost.

It is anticipated that combined sewage pumping stations will be required at four of the thirteen locations. Based on very preliminary studies, the costs of these four pumping stations will add approximately $8,500,000 to the above cost for detention tanks.

The use of detention tanks to receive flows from all of the combined sewer overflow points along the Milwaukee River also requires the construction of sewers for interconnecting the various outfalls tributary to each of the thirteen detention tanks. Based on preliminary studies, the costs for these interconnecting sewers are estimated to be approximately $[illegible],000,000.

The preliminary costs as discussed herein for a City-wide system of detention of combined sewer overflows represent only a part of the total economic impact of such a system. Treatment facilities would be strongly affected by detention and pump-out arrangements. The degree of effect would be a function of total storage volumes and pump-out rate.

Evaluation of the costs of treatment facilities as a consequence of a comprehensive detention system is beyond the scope of this Report. However, it can be concluded that the implementation of a combined sewer overflow storage plan for the City of Milwaukee will require a concurrent evaluation of treatment plant capacity.

This preliminary estimate is not based on a detailed feasibility study for each location. It is based on a general visual survey in the vicinity of each overflow. In that survey, general locations for detention tanks were determined based on available open space, degree of property improvements, and access to the location from the sewer system.

Several assumptions were made for estimating purposes for each location:

1. Connecting sewer sizes were assumed to be the same as the diameter of the overflow, or overflows, to be connected to each tank.

2. Average depth of connecting sewers was assumed to be about 15 feet in all cases.

3. Tank volume for each location was based on the sizing used for the Humboldt Avenue tank under the assumption that water quality criteria will be met with the removals possible with that sizing. That is, tank volume was adjusted upwards or downwards in proportion to the relationship between the area tributary to each site, and the effective area tributary to the Humboldt Avenue site. This approach also implies constant rainfall and runoff characteristics for all locations.

4. Tank construction costs for all locations were based on the unit costs associated with the Humboldt Avenue tank as presented in Section 11. Variation in foundation problems, and consequential effects on construction costs could not be considered.

A preliminary location of 13 detention tanks is indicated in Figure 80. Each tank is sized to receive the overflow from several of the 32 combined sewer outlets to the River. These outlets have an approximate total combined sewer overflow drainage area of approximately 5800 acres.

A listing of the thirteen detention tanks, the approximate combined sewer acreage which would contribute to each tank, tank size, and the estimated costs of the tanks are included in Table 48.

General cost information relating cost per square mile to percent removal of combined sewer overflow based on detention tanks has been developed. Based on one detention tank serving a 0.5 square mile area the capital cost per square mile varies from approximately $2,800,000 to $3,200,000 as percent removal increases from 50 to 80. Based on one tank serving a 2.0 square mile area the capital cost varies from $1,000,000 to $1,400,000 as percent removal increases from 50 to 80. This data is illustrated graphically in Figure 81.

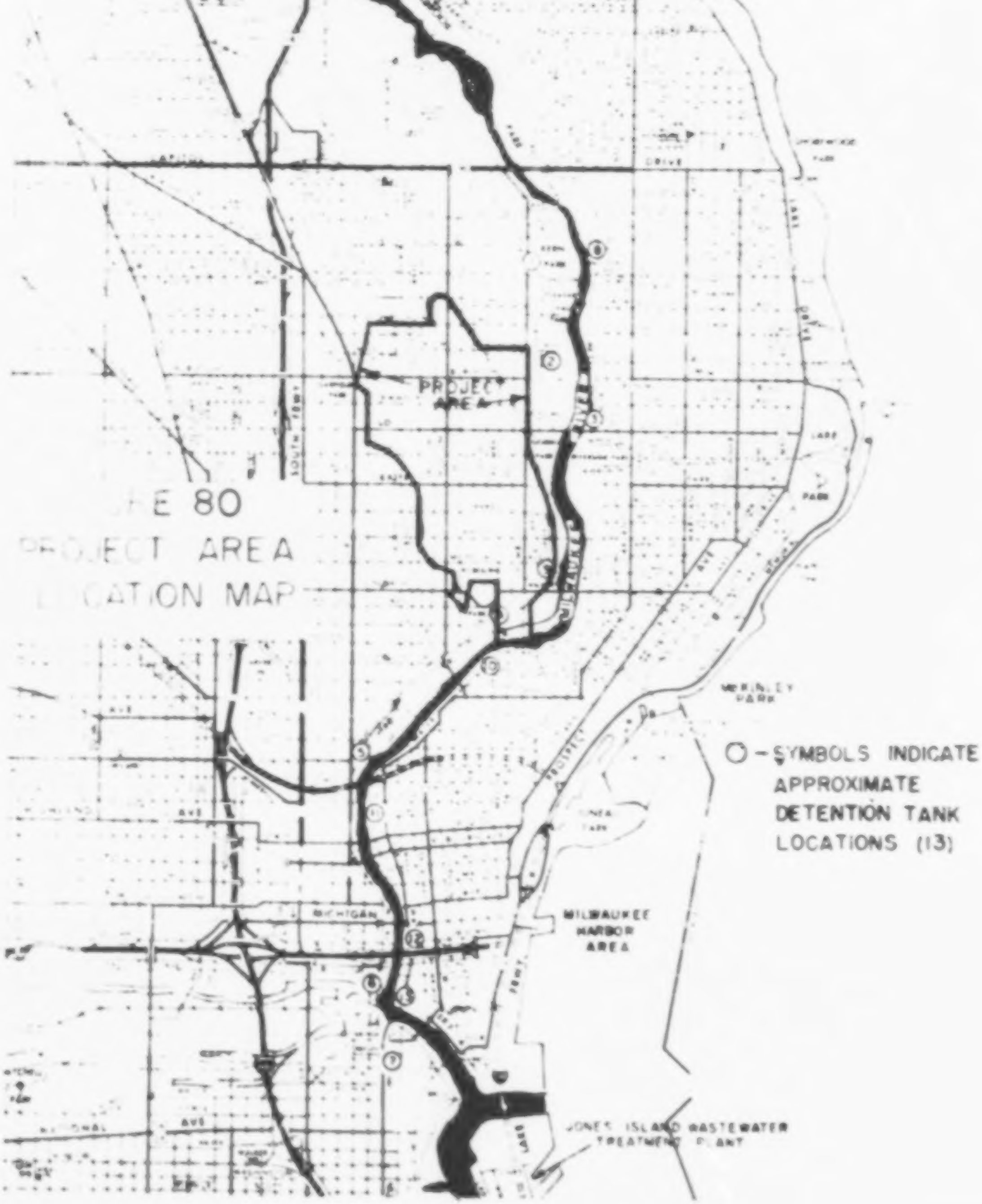


FIGURE 80
PROJECT AREA
LOCATION MAP

NORTH
PROJECT AREA
MILWAUKEE RIVER
LINCOLN PARK
ESTABROOK PARK
McKINLEY PARK
MICHIGAN
AVE
DRIVE
LAKE
SOUTH
MILWAUKEE HARBOR AREA
JONES ISLAND WASTEWATER TREATMENT PLANT
○ - SYMBOLS INDICATE APPROXIMATE DETENTION TANK LOCATIONS (13)

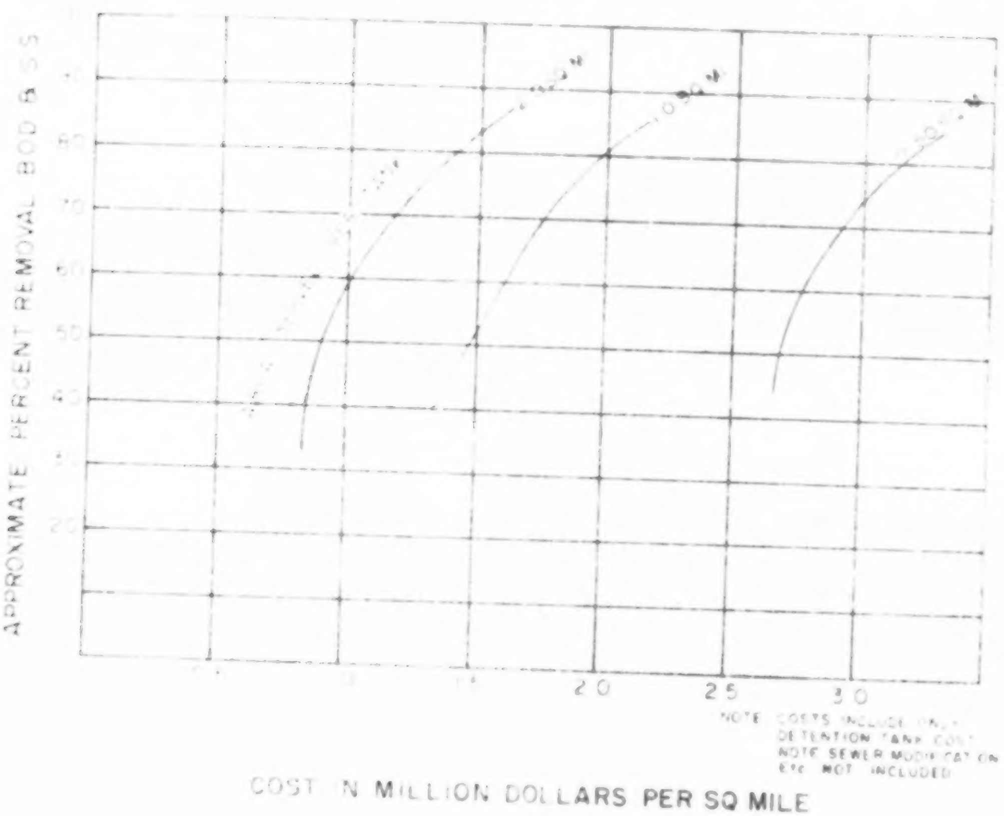
Table 49. MILWAUKEE RIVER DETENTION TANK DATA

Tank Number	Size of Contributing Area (Acres)	Tank Capacity (Million Gallons)	Estimated Cost
1	690	4.7	$ 2,600,000
2	610	4.2	2,500,000
3	320	2.2	2,100,000
4	570	3.9	2,400,000
5	800	5.8	2,400,000
6	420	2.9	2,200,000
7	95	0.7	1,800,000
8	755	5.2	2,600,000
9	855	5.9	3,000,000
10	200	1.4	2,000,000
11	210	1.4	2,000,000
12	205	1.4	2,000,000
13	95	0.7	1,800,000
		TOTAL	$29,400,000
		Use	$29,500,000

Based on December, 1973 prices.

FIGURE 81

COST PER UNIT AREA SERVED
FOR
DETENTION TANK CONSTRUCTION



NOTE COSTS INCLUDE ONLY DETENTION TANK COST NOTE SEWER MODIFICATION ETC NOT INCLUDED

SECTION XI

REFERENCES

1. Pollutional Effects of Storm Water and Overflows from Combined Sewer Systems, Public Health Service Publication No. 1246. Washington, D.C., 1964.

2. Sullivan, R.H. Assessment of Combined Sewer Problems. Combined Sewer Overflow Abatement Technology, U.S. Federal Water Quality Administration. Chicago, Illinois, U.S. Government Printing Office, June 1970. Pages 107-121.

3. The Milwaukee River - An Inventory of Its Problems, An Appraisal of Its Potential. Milwaukee River Technical Study Committee. Milwaukee, Wisconsin. 1968.

4. Standard Methods For the Examination of Water and Wastewater. 13th Edition. American Public Health Association, Inc. 1971.

5. Geldreich, E.E., Clark, H.F., Huff C.B. and Best, L.C. Fecal-Coliform Organism Medium for the Membrane Filter Technique. Journal of AWWA. Vol. 57: Pages 208-214, 1965.

SECTION XII

APPENDICES*

I. Description of Detention Tank Model

II. General River Water Quality Data

III. Seasonal, Hourly, Diurnal Variation In River Quality Data

IV. Dry Weather Sewage Quality Data

V. Wet Weather Sewage Quality Data

VI. Rainfall Frequency Histogram - Station 1 Data 1949 - 1964

VII. Sewage Flow Data

VIII. Rainfall Data

*Note: Appendix I is included as part of this Report. Other Appendices are not included but are on file with the City of Milwaukee, Department of Public Works.

APPENDIX I

DESCRIPTION OF DETENTION STORM TANK MODEL

General Comments

A mathematical computer model was developed to describe the performance characteristics of a storm detention tank. The computer model consists of a main program and five sub-programs. A listing of these programs is given later in the Appendix. A description of all the programs is given below. A computer printout of the program is included following the program descriptions.

Main Program

The main program is responsible for coordinating the activities of all sub-programs and reading in and printing out all the necessary data to assess the performance of the storm detention tank. The program operates on hourly rainfall data and calculates the quality and quantity of runoff generated. A comparison of the quantity of runoff plus the normal dry weather flow is made with the interceptor capacity. If this capacity has not been exceeded, all the combined sewage is retained in the sewer system and directed to the treatment plant. When interceptor capacity is exceeded, the excess portion is directed to the storm detention tank. During the intervals when it is not raining a certain amount of the tank contents, depending on the pump out rate, is returned to the treatment plant. If the runoff is greater than the interceptor capacity and storm detention tank capacity, the excess is discharged to the stream. The user of the model has the choice of determining two types of tank performance, bypass or plug flow. In plug flow each hourly increment retains its identity and the first element into the tank is the first one out of the tank. In a bypass assumption , the tank is circumvented when full. The model keeps track of the following items which are necessary to assess performance: hourly rainfall, quantity and quality both to the tank and to the stream, number of storms with flow to tank and to the stream, number of hours of flow to tank and to the stream, and the total number of hours of rainfall.

Following a description of the various subprograms an illustration of the input in appropriate format is presented.

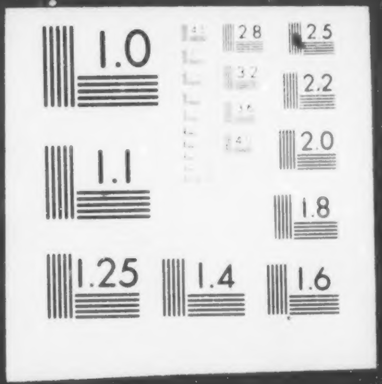
Subroutine Sread

This program is used when there is both a volume of storm water in the detention tank at the onset of the event under investigation and when the tank performance characteristics are assumed to be plug flow. The purpose of this subroutine is to read into the computer the initial contents of the detention tank. The initial contents are read in the following manner. The first card gives the number of "layers" present in the detention tank at the outset. A layer is defined as an hourly increment of inflowing storm water. Since the characteristics are plug flow no mixing occurs and each increment of wastewater must maintain its own identity. The succeeding cards describe the characteristics of each layer. One card is used for each layer. The following information is contained on each card: quantity of layer in million gallons, strength of contaminant in pounds, and time duration of layer in hours. When the contents are completely read and checked, control is transferred to the main program.

4 OF 4

PB

250 427


1.0
2.8
2.5
3.2
2.2
3.6
4.0
2.0
1.1
1.8
1.25
1.4
1.6

Subroutine Enter

This program is used when tank performance is assumed to be plug flow and either one of the following conditions exist: it has rained during the past hour and the interceptor capacity has been exceeded, or it has rained during the past hour and the interceptor capacity has not been exceeded but the tank was not empty prior to the past hour. The purpose of this subroutine is to enter into the computer memory the new "layer" if it has rained during the last hour and update information relative to old layers still held in the detention tank. This new layer data is calculated in the main program and a read statement is not necessary. The quantity of the new layer is obtained by subtracting the interceptor capacity from the sum of the wet weather and dry weather sewer flow. The dry weather flow is an inputted variable and wet weather flow is determined by calculation from the rainfall drainage area, and runoff coefficient. In this subprogram, the tank will accept the total volume of new input, even in cases where it may be full. The amount (pounds) of contaminant in the new layer is determined by calculation from the volume and concentration which is an inputted variable. The third identifying characteristic (time duration) of this layer is set to zero hours. In addition the subroutine increases by 1 hour the time duration of all old layers withheld in the detention tank. When these two functions are complete control is transferred to the main program.

Subroutine Outof

This program is used when tank performance is assumed to be plug flow and the volume entered into the detention tank has exceeded the tank capacity. It should be recalled that a capacity check was not made in subroutine enter but rather this subroutine accepted any volume which was entered. The purpose of this program is therefore, to calculate the strength and quantity of wastewater that cannot be retained in the tank. An impliction of this plug flow is that the first layer to enter the tank is the first layer to leave when capacity has been exceeded. Since averages are not computed, the calculation of the total flow being discharged without regard to treatment is straightforward. It is merely the sum of the quantities of each layer minus the capacity of the tank. The total flow leaving the tank must be compared with a summation of flow from each layer added one at a time to determine how many layers and/or a portion of a layer are/is being forced out of the tank. The determination of the total pounds discharged to the river is more involved however. Associated with each layer portion thereof is an identifiable strength in pounds. Contaminant removal in the storm tank is due to two factors, settling and detention. Retention can be defined as volume which is prevented from overflowing to the river due to excess tank availability. In addition, each layer has associated with it a certain removal due to settling, the amount dependent on the time interval in the tank. The fraction of each layer removed is calculated from the equation $R = A(1 - e^{-kt})$ where A and k are inputted variables for each specific contaminant, and t is the time which is continually updated for each layer. Accordingly, this equation is applied to every layer or portion thereof when it is discharged. The fraction removed by settling is assumed to be removed continuously from the tank. The fraction not removed by sedimentation is the strength discharged to the river when the tank capacity is exceeded. The total strength therefore is the summation of the individual strengths associated with the discharged quantities.

This program in addition keeps track of the number of layers at any specific time and also the characteristics of each layer which are flow, quality, and residence time in the tank. Upon completion of these tasks control is transferred to the main program.

Subroutine Withd

This program is used when tank performance is assumed to be plug flow and the following condition exists: it has not rained during the past hour and the tank is not empty. The purpose of this program is to withdraw an amount of wastewater during dry periods and route it to the treatment plant. The amount withdrawn hourly is held constant (even though in practice it may be varied) and is calculated from an inputted variable describing the number of hours to pump out the tank contents. The flow leaving the tank is compared with a summation of flows from each layer added one at a time to determine how many layers and/or a portion of a layer is/are being returned to the treatment plant. Associated with each layer/or portion thereof is an identifiable strength in pounds. The total strength withdrawn is a summation of the individual strengths associated with the discharged quantities. This program in addition keeps track of the number of layers remaining at any specific time, and also the characteristics of each layer which are flow, quality, and residence time. When the remaining layers are updated for time duration, control is transferred to the main program.

Subroutine Head

This program is used to print up column headings and keep track of margins and lines remaining on a page. It also points up the date and counts the pages of output.

Illustration of Fortran Card Input to Computer Model

Card Number	Column	Variable	Description and Comment
1	1-5	NSET	Lateger variable right justified. Number of sets of data. A set can be no greater than 366 days but may be less. The sets must be consecutive.
1	6-10	TYPE	Integr variable right justified. It is a description of tank performance. If equal to 1 plug flow is assumed. If equal to 2 bypass is assumed.
2	1-10	ILAST	Integr variable right justified. Time interval in hours from the last storm to the beginning of the period under consideration.

Card Number	Column	Variable	Description and Comment
2	11-20	EMPT	Integr Variable right justified. Number of hours it takes to empty the retention tank.
2	21-30	AREA	Floating Point Variable. Drainage area in acres.
2	31-40	CAPIN	Floating point variable. The capacity of the interceptor system in million gallons per hour.
2	41-50	TANKC	Floating point variable. The tank capacity in million gallons.
2	51-60	TANK	Floating point variable. The quantity of combined sewage at the beginning of the period under consideration in million gallons.
3	1-10	DWF(1)	Floating point variable. Average dry weather sewer flow at 1:00AM.
3	11-20	DWF(2)	Floating point variable. Average dry weather sewer flow at 2:00AM.
3	21-30	DWF(3)	Floating point variable. Average dry weather sewer flow at 3:00AM.
3	31-40	DWF(4)	Floating point variable. Average dry weather sewer flow at 4:00AM.
3	41-50	DWF(5)	Floating point variable. Average dry weather sewer flow at 5:00AM.

Card Number	Column	Variable	Description and Comments
3	51-60	DWF(6)	Floating point variable. Average dry weather sewer flow at 6:00AM.
3	61-70	DWF(7)	Floating point variable. Average dry weather sewer flow at 7:00AM.
3	71-80	DWF(8)	Floating point variable. Average dry weather sewer flow at 8:00AM.
4	1-10	DWF(9)	Floating point variable. Average dry weather sewer flow at 9:00AM.
4	11-20	DWF(10)	Floating point variable. Average dry weather sewer flow at 10:00AM.
4	21-30	DWF(11)	Floating point variable. Average dry weather sewer flow at 11:00AM.
4	31-40	DWF(12)	Floating point variable. Average dry weather sewer flow at noon.
4	41-50	DWF(13)	Floating point variable. Average dry weather sewer flow at 1:00PM.
4	51-60	DWF(14)	Floating point variable. Average dry weather sewer flow at 2:00PM.
4	61-70	DWF(15)	Floating point variable. Average dry weather sewer flow at 3:00PM.

Card Number	Column	Variable	Description and Comments
4	71-80	DWF(16)	Floating point variable. Average dry weather sewer flow at 4:00PM.
5	1-10	DWF(17)	Floating point variable. Average dry weather sewer flow at 5:00PM.
5	11-21	DWF(18)	Floating point variable. Average dry weather sewer flow at 6:00PM.
5	21-30	DWF(19)	Floating point variable. Average dry weather sewer flow at 7:00PM.
5	31-40	DWF(20)	Floating point variable. Average dry weather sewer flow at 8:00PM.
5	41-50	DWF(21)	Floating point variable. Average dry weather sewer flow at 9:00PM.
5	51-60	DWF(22)	Floating point variable. Average dry weather sewer flow at 10:00PM.
5	61-70	DWF(23)	Floating point variable. Average dry weather sewer flow at 11:00PM
5	71-80	DWF(24)	Floating point variable. Average dry weather sewer flow at Midnight.
6	1-10	C	Floating point variable. Runoff coefficient.

Card Number	Column	Variable	Description and Comments
6	11-20	WC(1)	Floating point variable. Average concentration of contaminant during first hour of storm.
6	21-30	WC(2)	Floating point variable. Average concentration of contaminant during second hour of storm.
6	31-40	WC(3)	Floating point variable. Average concentration of contaminant during third hour of storm.
7	1-80	HED(1)- HED(80)	Aphanumeric format. Any suitable title for the first set of data.
8	1-2	MONT2	Integer variable right justified. Ending month for 1st set of data.
8	4-5	DAY2	Integer variable right justified. Ending day for 1st set of data.
8	7-10	YEAR2	Integer variable right justified. Ending year for 1st set of data.
8	21-22	MONT1	Integer variable right justified. Beginning month for 1st set of data.
8	24-25	DAY1	Integer variable right justified Beginning day for 1st set of data.
8	27-30	YEAR1	Integer variable right justified. Beginning year for 1st set of data.

Card Number	Column	Variable	Description and Comments
9	1-10	PCENT	Floating point variable. It is the asymptote in the settling equation $R=PCENT\ (1 - e^{-Kt})$ which defines tank performance in 1st. It is expressed as a percent.
9	11-20	RATE	Floating point variable. It is the "k" term in the same settling equation. The units are (1/hour).
10	1-5	LAYER	Integer variable. When the performance of tank is assumed to be plug flow and the tank is not empty at the beginning of the set of data the following cards are required. Layer to the number of plug flow elements initially in the tank.
11	1-10	FLOW(1)	Floating point variable. Quantity of flow in million gallons of first elemental layer.
11	11-20	POWL(1)	Floating point variable. Pounds of contaminant in first elemental layer.
11	21-30	ITIME(1)	Integer variable right justified. Time duration in hours of first elemental layer.
12 and following:			One card is repeated for each additional elemental layer. The format is the same as Card 11.

Card Number	Column	Variable	Description and Comments
13	7-8	YEAR	Integer variable right justified. Year rainfall occurred during 1st set of data
13	9-10	MONTH	Integer variable right justified. Month rainfall occurred during 2nd set of data
13	11-12	DAY	Integer variable right justified. Day rainfall occurred during 3rd set of data.
13	13	LSEQ	Describes what part of day rainfall occurred LSEQ=1, it is AM LSEQ=2, it is PM.
13	14-16	IHR(1)	Rainfall during first hour expressed in .01 inches.
13	17-19	IHR(2)	Rainfall during second hour expressed in .01 inches.
13	20-22	IHR(3)	Rainfall during third hour expressed in .01 inches.
13	23-25	IHR(4)	Rainfall during forth hour expressed in .01 inches.
13	26-28	IHR(5)	Rainfall during fifth hour expressed in .01 inches.
13	28-31	IHR(6)	Rainfall during sixth hour expressed in .01 inches.
13	32-34	IHR(7)	Rainfall during seventh hour expressed in .01 inches.

Card Number	Column	Variable	Description and Comments
13	35-37	IHR(8)	Rainfall during eighth hour expressed in .01 inches.
13	38-40	IHR(9)	Rainfall during ninth hour expressed in .01 inches.
13	41-43	IHR(10)	Rainfall during tenth hour expressed in .01 inches.
14	44-46	IHR(11)	Rainfall during eleventh hour expressed in .01 inches.
13	47-49	IHR(12)	Rainfall during twelth hour expressed in .01 inches.
14 and following:			As many cards as necessary are used to read into the computer all the rainfall during the first set of data. Same format as Card 13. It is important that a card be used only when there is rainfall for at least one or more hours in the twelve hour period. A blank in columns 7 and 8 signifies the end of rainfall record in this set. Therefore a blank card must follow the last rainfall card.

If there is more than one set of data repeat cards seven and following for each set.

```
      DEFINE FILE 1 (366,27,U,IXY)
      INTEGER OUT,PAGE,YR,MO,DAY,EMPT
      INTEGER TYPE,YEAR1,YEAR2,MONT1,MONT2,DAY1,DAY2
      DIMENSION WC(5),INPHR(27),HED(20),IHR(12),DWF(24)
      COMMON FLOW(100),POUND(100),ITIME(100)
   17 FORMAT(2(I2,1X,I2,1X,I4,10X))
   18 FORMAT(8F10.2)
 1000 FORMAT(2I5)
 2000 FORMAT(2I10,6F10.3)
 2001 FORMAT(1H1,//,10X,'THE TIME INTERVAL SINCE THE LAST STORM IS',I8,3
     1X,'HOURS',/,10X,'IT TAKES',I8,3X,'HOURS TO EMPTY THE TANK',/,10X,'
     2THE DRAINAGE AREA IS',F10.0,3X,'ACRES',/,10X,'THE CAPACITY OF THE
     3INTERCEPTOR IS',F10.1,3X,'MIL.GAL./HOUR  ',/,10X,'THE CAPACITY OF
     4THE STORM RETENTION TANK IS',F10.1,3X,'MILLION GALLONS')
 2003 FORMAT(10X,'THE ' ,
     2'VOLUME OF STORM WATER IN THE TANK AT TIME ZERO IS',F10.1,3X,'MILL
     3ION GALLON')
 2002 FORMAT(                                  10X,'THE RUNOFF COEFFICIENT IS',F10.
     12,/,10X,'THE BOD CONCENTRATION DURING THE FIRST HOUR IS',F10.0,3X,
     2'MG/L',/,10X,'THE BOD CONCENTRATION DURING THE SECOND HOUR IS',F10
     3.0,3X,'MG/L',/,10X,'THE BOD CONCENTRATION AFTER TWO HOURS IS',F10.
     40,3X,'MG/L')
 2006 FORMAT(10X,'TANK PERFORMANCE IS ASSUMED TO BE PLUG FLOW')
 2007 FORMAT(10X,'TANK PERFORMANCE IS ASSUMED TO BE BYPASS')
 3000 FORMAT(6X,3I2,I1,12I3)
 4000 FORMAT(20X'NUMBER OF STORMS WITH FLOW      TANK=')
 4001 FORMAT('+'47X'TO'7X,I4)
 4002 FORMAT(21X'NUMBER OF HOURS WITH FLOW       TANK=')
 4003 FORMAT('+'46X'FROM'6X,I4)
 4004 FORMAT(20X'NUMBER OF HOURS WITH RAINFALL ='I5)
 5000 FORMAT('  STORM TOTALS  '45('.'),2(F8.3,F11.0,1X),F7.3)
 5001 FORMAT('  SUB   TOTALS  '45('-'),2(F8.3,F11.0,1X),F7.3)
 5002 FORMAT('  GRAND TOTALS  '45('*'),2(F8.3,F11.0,1X),F7.3)
 6000 FORMAT(I4,I3'-'I2'-'I2)
 7000 FORMAT(1H+,14X,F4.2,3X,F8.3,2X,F9.0,1X,F8.3,2X,F8.0)
 8000 FORMAT(20A4)
      INP=2
      OUT=3
      PAGE=0
      LINE=60
```

```
C      INITIALIZE GRAND TOTALS
       TQI=0.0
       TQE=0.0
       TWLT=0.0
       TWLS=0.0
C      INITIALIZE STORM TOTALS
       QI=0.0
       QE=0.0
       WLT=0.0
       WLS=0.0
C      QI = FLOW INTO STORM RETENTION TANK
C      QE = FLOW FROM RETENTION TANK TO RIVER
C      WLT = BOD INTO STORM RETENTION
C      WLS = BOD FROM STORM RETENTION TANK TO RIVER
ALS    IF THE ABOVE FOUR VARIABLES ARE PREFIXED BY A T  THEY REFER TO TOT
   C        IF PREFIXED BY AN S  THEY REDER TO SUBTOTALS
C      INITIALIZE COUNTERS
       NOW=0
       IST=1
C      IST=CUMULATOR FOR THE NUMBER OF STORMS
C      NOW=CUMULATOR FOR HOURS IT HAS RAINED DURING A PARTICULAR STORM
       READ(INP,1000)NSET    ,TYPE
C      NSET = NUMBER OF SETS OF DATA  A SET CAN BE NO GREATER THAN 366
C      DAYS BUT MAY BE LESS. THE SETS MUST BE CONSECUTIVE
C      TYPE=DESCRIPTION OF TANK CHARACTERISTICS
C      IF 1   IT IS PLUG FLOW
C      IF 2     IT IS BYPASS
C      IF 3       IT IS COMPLETELY MIXED
       READ(INP,2000)ILAST,EMPT,AREA,CAPIN,TANKC,TANK
       READ(INP,18)(DWF(I),I=1,24)
       READ(INP,18)C,(WC(I),I=1,3)
C    ILAST = TIME INTERVAL SINCE LAST STORM(HOURS)
C    EMPT = NUMBER OF HOURS IT TAKES TO EMPTY THE TANK
C    AREA = DRAINAGE AREA(ACRES)
C    CAPIN = INTERCEPTOR SEWER CAPACITY(MG/HR)
C    TANKC    = CAPACITY OF THE TANK( 4 MG)
C TANK IS QUANTITY OF WATER(MG) IN TANK AT THE BEGINNING OF RUN
C    DWF = AVERAGE DR  WEATHER SEWER FLOW(MG/HR)
```

```
C    C = RUNOFF COEFFICIENT
OURSWC(I) ARE AVERAGE BOD VALUES OF ALL THE STORMS FOR THE FIRST THREE H
           WRITE(OUT,2001) ILAST,EMPT,AREA,CAPIN,TANKC
      WRITE(OUT,2002) C,(WC(I),I=1,3)
      WRITE(OUT,2003) TANK
      GO TO (2004,2005),TYPE
 2004 WRITE (OUT,2006)
      GO TO 2008
 2005 WRITE (OUT,2007)
 2008 CONTINUE
C    XXX = AMOUNT OF FLOW(MG) PUMPED OUT OF TANK PER HOUR
      XXX=TANKC/EMPT
      DO 999 NSETN=1,NSET
      READ(INP,8000) (HED(I),I=1,20)
C     READ ENDIMG DATE IN COLUMNS U-10 AND BEGINNING DATES IN 21-30
      READ (INP,17)MONT2,DAY2,YEAR2,MONT1,DAY1,YEAR1
      READ (INP,18)  PCENT,RATE
C     RATE= K RATE COEFFICIENT IN THE SETTLING EQUATION
C     PCENT=EFFICIENCY OF STORM TANK SETTLING PERFORMANCE
C     INITIALIZE SUBTOTALS
      SQI=0.0
      SQE=0.0
      SWLT=0.0
      SWLS=0.0
      NSTQI=0
      NHRQI=0
      NSTQE=0
      NHRQE=0
      NRAIN=0
C     NSTQI=NUMBER OF STORM EVENTS WHEN RUNOFF EXCEEDED SEWER CAPACITY
C     NHRQI= NUMBER OF HOURS WHEN RUNOFF EXCEEDED SEWER CAPACITY
CEEDEDNSTQE= NUMBER OF STORM EVENTS WHEN THE STORM RETENTION TANK WAS EX
      C      NUMBER OF HOURS WHEN THE STORM RETENTION TANK WAS EXCEEDED
C     NRAIN=NUMBER OF HOURS IT RAINED
      LAYER=0
      TPOUN=0.
C     LAYER=NUMBER OF PLUG FLOW ELEMENTS WITHIN THE TANK
C     TPOUN= TOTAL POUNDS OF MATERIAL IN TANK AT START
      GO TO (31,32),TYPE
```

```
31 IF (TANK  GT  0 ) CALL SPEAD (TANK ILAST TPOUN LAYER)
32 CONTINUE
   INUM=IDATE(DAY1,MONT1,YEAR1) -1
   DO 10 I=1,27
10 INPHR(I)=0
   DO 11 I=1,366
11 WRITE(1'I)(INPHR(J),J=1,27)
20 READ(INP,3000) YR,MO,DAY,ISEQ,(IHR(J),J=1,12)
   IF(YR LE 0) GO TO 25
   YR=1900+YR
   I=IDATE(DAY,MO,YR)-INUM
   READ(1'I)(INPHR(J),J=1,27)
   INPHR(1)=DAY
   INPHR(2)=MO
   INPHR(3)=YR
   J=0
   IF(ISEQ EQ 2)J=12
   DO 16 K=1,12
   L=K+J+3
16 INPHR(L)=IHR(K)
   WRITE(1'I)(INPHR(J),J=1,27)
   GO TO 20
25 NUM=IDATE(DAY2,MONT2,YEAR2)-INUM
   DO 200 N=1,NUM
   READ(1'N)DAY,MO,YR,(INPHR(I),I=1,24)
   DO 200 I=1,24
   IF(INPHR(I) GT 0) GO TO 60
   IF (DAY  NE  0) ILAST=ILAST+1
   IF(DAY EQ 0) ILAST=ILAST+24
   XX=XXX
   IF(DAY EQ 0) XX=24 *XXX
   IF(DAY EQ 0) I=24
   INTVL=1
   IF(DAY  EQ  0 ) INTVL=24
   GO TO (43,44),TYPE
44 TANK=TANK-XX
   IF(TANK LT 0 ) TANK=0
   GO TO 45
43 IF (TANK  GT  0 ) CALL WITHD (TANK,ILAST,TPOUN,LAYER,XX,INTVL)
```

```
   45 IF(NOW)200,200,51
   51 IF(ILAST.LE.2) GO TO 200
      REM=(WLT-WLS)/WLT
      CALL HEAD(PAGE,LINE,HED)
      WRITE(OUT,5000) QI,WLT,QE,WLS,REM
      IF (QI .GT. 0.001) NSTQI=NSTQI+1
      IF (QE .GT. 0.001) NSTQE=NSTQE+1
C     COMPUTE SUBTOTALS
   58 SQI=SQI+QI
      SQE=SQE+QE
      SWLT=SWLT+WLT
      SWLS=SWLS+WLS
C     INITIALIZE STORM TOTALS
      QI=0.0
      QE=0.0
      WLT=0.0
      WLS=0.0
      IST=IST+1
      NOW=0
      GO TO 200
   60 NOW=NOW+1
      NRAIN=NRAIN+1
      III=NOW
      IF(NOW.GE.3) III=3
      RUNOF=INPHR(I)*AREA*C*2.71142E-04
      QTEMP=RUNOF+DWF(I)-CAPIN
      IF (QTEMP .LT. 0.) QTEMP=0
      PTEMP=QTEMP*WC(III)*8.34
      GO TO(63,64),TYPE
   64 TANK=TANK+QTEMP
      GO TO 160
   63 IF (TANK .LE. 0. .AND. QTEMP .LE. 0.) GO TO 160
      CALL ENTER (TANK,TPOUN,LAYER,QTEMP,PTEMP)
  160 CONTINUE
      EXITQ=TANK-TANKC
      IF (EXITQ .LT. 0.) EXITQ=0
      IF (EXITQ .LE. 0.) EXITP=0
      IF (EXITQ .LE. 0.) GO TO 150
      TANK=TANKC
      GO TO (73,74),TYPE
   74 EXITF=EXITQ*WC(I11)*8.34
      GO TO 150
   73 CALL OUTOF (EXITQ,LAYER,EXITF,IFOUN,PCENT,RATE)
  150 CONTINUE
  190 RAIN=INPHR(I)/100.+0.005
      CALL HEAD(PAGE,LINE,HED)
      YR=YR-1900
      WRITE(OUT,6000)IST,MO,DAY,YR
      YR=YR+1900
      WRITE (OUT,7000) RAIN,QTEMP,FTEMP,EXITQ,EXITF
      IF (QTEMP .GT. 0.001) NHRQI=NHRQI+1
      IF (EXITQ .GT. 0.001) NHRQE=NHRQE+1
      QI=QI+QTEMP
      WLT=WLT+FTEMP
      QE=QE+EXITQ
      WLS=EXITF+WLS
      ILAST=0
  200 CONTINUE
      REM=(SWLT-SWLS)/SWLT
      CALL HEAD(PAGE,LINE,HED)
      WRITE(OUT,5001)SQI,SWLT,SQE,SWLS,REM
      CALL HEAD(PAGE,LINE,HED)
      WRITE(OUT,4000)
      WRITE(OUT,4001)NSTQI
      CALL HEAD(PAGE,LINE,HED)
      WRITE(OUT,4002)
      WRITE(OUT,4001)NHRQI
      CALL HEAD(PAGE,LINE,HED)
      WRITE(OUT,4000)
      WRITE(OUT,4003)NSTQE
      CALL HEAD(PAGE,LINE,HED)
      WRITE(OUT,4002)
      WRITE(OUT,4003)NHRQE
      CALL HEAD(PAGE,LINE,HED)
      WRITE(OUT,4004)NRAIN
      LINE=60
      TQ1=TQ1+SQ1
      TQE=TQE+SQE
      TWLT=TWLT+SWLT
      TWLS=TWLS+SWLS
  999 CONTINUE
      REM=(TWLT-TWLS)/TWLT
      CALL HEAD(PAGE,LINE,HED)
      WRITE(OUT,5002)TQ1,TWLT,TQE,TWLS,REM
      CALL EXIT
      END
      SUBROUTINE SPEAD(TANK,ILAST,TPOUN,LAYER)
      INTEGER OUT
      COMMON FLOW(100),POUND(100),ITIME(100)
      INP=2
      OUT=3
   10 FORMAT(I5)
   11 FORMAT(2F10.0,I10)
   12 FORMAT ('THIS IS AN ERROR MESSAGE')
C     READ INITIAL CONTENTS OF TANK
      READ(INP,10) LAYER
      QTEMP=0
      TPOUN=0
      DO 3 I=1,LAYER
      READ(INP,11) FLOW(I),POUND(I),ITIME(I)
      TPOUN=TPOUN+POUND(I)
    3 QTEMP=QTEMP+FLOW(I)
      IF((QTEMP-TANK).GT.(.01*TANK))WRITE(OUT,12)
      IF(ITIME(LAYER).NE.ILAST) WRITE(OUT,12)
      RETURN
      END
      SUBROUTINE WITHD (TANK,ILAST,TPOUN,LAYER,XXX,INTVL)
      INTEGER OUT
      COMMON FLOW(100),POUND(100),ITIME(100)
      INP=2
      OUT=3
      QTEMP=0
      DO 5 I=1,LAYER
      QTEMP=QTEMP+FLOW(I)
      IF (QTEMP.GT.XXX) GO TO 6
    5 CONTINUE
      TANK=0
      DO 8 I=1,LAYER
      FLOW (I)=0
      POUND(I)=0
    8 ITIME (I)=0
      LAYER=0
      TPOUN=0
      RETURN
    6 J=I-1
      IF (J.EQ.0)OJ=0
      IF (J .GT. 0) OJ=OTEMP-FLOW(I)
      RATIO=(XXX-OJ)/FLOW(I)
      FLOW(I)=FLOW(I)*(1.-RATIO)
      POUND(I)=POUND(I)*(1.-RATIO)
      TANK=TANK-XXX
      IF(I.EQ.1) GO TO 9
      DO 7 K=I,LAYER
      L=K-J
      FLOW(L)=FLOW(K)
      POUND(L)=POUND(K)
      ITIME(L)=ITIME(K)
    7 CONTINUE
      LAYER=LAYER-J
    9 TPOUN=0
      DO 14 I=1,LAYER
      ITIME(I)=ITIME(I) + INTVL
   14 TPOUN=TPOUN+POUND(I)
      RETURN
      END
      SUBROUTINE HEAD(PAGE,LINE,HED)
      INTEGER PAGE
      DIMENSION HED(1)
      LINE=LINE+1
      IF(LINE-50)20,10,10
10    LINE=0
      PAGE=PAGE+1
      WRITE(3,100)(HED(I),I=1,20),PAGE
      WRITE(3,200)
      WRITE(3,300)
      WRITE(3,400)
100   FORMAT('1'19X,20A4,14X'PAGE'I3/)
200   FORMAT('   STORM         RAIN      FLOW TO    BOD TO    FLOW FROM ,
     2 '    BOD TO    ','WAT',
     3'ER TO    BOD TO  WATER FROM  BOD TO    REMOVAL')
300   FORMAT('    NUMBER'15X'TANK         TANK        TANK'
     2,'    STREAM         TANK         TANK    ',
     1'    TANK      STREAM')
400   FORMAT(12X'(IN/HR)   (MG/HR)    (LBS/HR)   (MG/HR)    LBS/HR
     2'(MG)      (LBS)        (MG)        (LBS)')
20    RETURN
      END
      SUBROUTINE ENTER(TANK,TPOUN,LAYER,QTEMP,PTEMP)
      COMMON FLOW(100),POUND(100),ITIME(100)
      IF(QTEMP .LE. 0 ) J=LAYER
      IF(QTEMP .LE. 0 ) GO TO 14
      LAYER=LAYER+1
      FLOW(LAYER)=QTEMP
      TANK=TANK+QTEMP
      POUND(LAYER)=PTEMP
      TPOUN=TPOUN+PTEMP
      ITIME(LAYER)=0
      J=LAYER-1
      IF (J .EQ. 0) RETURN
   14 DO 13 I=1,J
   13 ITIME(I)=ITIME(I)+1
      RETURN
      END
      SUBROUTINE OUTOF (EXITQ,LAYER,EXITP,TPOUN,PCENT,RATE)
      INTEGER OUT
      COMMON FLOW(100),POUND(100),ITIME(100)
      OUT=3
      INP=2
      QTEMP=0
      DO 5 I=1,LAYER
      QTEMP=QTEMP+FLOW(I)
      IF (QTEMP .GT. EXITQ)  GO TO 6
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    5 CONTINUE
      WRITE (OUT,12)
   12 FORMAT (  THIS IAN ERROR MESSAGE )
      RETURN
    6 J=I-1
      IF (J EQ 0)OJ=0
      IF (J  GT  0) OJ=OTEMP-FLOW(I)
      RATIO=(EXITO-OJ)/FLOW(I)
      FLOW(I)=FLOW(I)*(1 -RATIO)
      EXITP=RATIO*POUND(I)
      RMAIN=1 -(PCENT/100 )*(1 -EXP(-RATE*ITIME(I)))
      DUMMY=EXITP*RMAIN
      POUND(I)=POUND(I)*(1 -RATIO)
      IF(I EQ 1) GO TO 9
      DO 8 K=1,J
      RMAIN=1 -(PCENT/100 )*(1 -EXP(-RATE*ITIME(K)))
      DUMMY=DUMMY+RMAIN*POUND(K)
    8 EXITP=EXITP+POUND(K)
      DO 7 K=I,LAYER
      L=K-J
      FLOW(L)=FLOW(K)
      POUND(L)=POUND(K)
      ITIME(L)=ITIME(K)
    7 CONTINUE
      LAYER=LAYER-J
    9 TPOUN=TPOUN-EXITP
      EXITP=DUMMY
      RETURN
      END
PIP>
```